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EARTHQUAKE ENGINEERING RESEARCH LABORATORY

EARTHQUAKE-RESISTANT DESIGN OF HIGH-RISE BUILDINGS

by
George W. Housner

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FOREWORD

Earthquake-resistant design requirements were first included in building codes in the United States in 1933, following the March 10, 1933 earthquake. At that time very little was known about destructive earthquake ground shaking and appropriate methods of designing structures to resist it. From 1933 to 1973 there has been a remarkable development in knowledge about earthquakes and about earthquake-resistant design. At the beginning of 1933, destructive earthquake ground shaking had never been recorded and, therefore, the nature of the ground shaking that caused earthquake damage was completely unknown, and the methods employed for earthquake-resistant design were extremely primitive and crude. During the forty years that have elapsed since 1933 research on earthquake engineering has developed much information of direct practical value. Instruments have been developed, and destructive earthquake ground motions have been recorded. Instruments installed in buildings have recorded their vibrations during strong earthquakes. Powerful methods of mathematical analysis have been developed for investigating the stresses and strains produced in buildings during earthquakes. The construction of high-rise buildings in Los Angeles affords an excellent example of how the results of research are put into practical application. In 1933 the building laws restricted the heights of buildings in Los Angeles to 13 stories,or less; whereas in 1973 there are buildings having as many as 60 stories. These modern high-rise buildings in Los Angeles were designed by advanced methods of dynamic analysis which were the results of research in earthquake engineering; and this report presents a brief account of how the results of the research finally ended up included in the 1973 Los Angeles Building Code.

Code Changes to Cut Quake Damage Passed

Council Unanimously OKs Tighter Rules for All New Building

BY ERWIN BAKER
Times City Bureau Chief

Changes in Los Angeles' building code designed to reduce earthquake damage in new construction by 80% were approved by the Los Angeles City Council Tuesday.

Building and Safety Department Supt. Robert J. Williams said the effect of the council's 13-0 vote would be a "substantial tightening of earthquake design."

The revision applies to all new construction—residential, commercial and industrial, including high-rise—after the effective date of the ordinance, which was sent to Mayor Sam Yorty for his signature or veto.

It also supplements the new quake-resistant criteria for the design of wood-frame construction and chimneys approved by the council last year.

All the changes—this year's and last year's—are based on studies of the damage caused by the disastrous February, 1971, earthquake.

The changes approved by the council Tuesday provide that:

—Buildings must be designed to withstand massive earthquakes.

Stresses to Be Reduced

—Stresses in some building materials, such as concrete, masonry and steel, must be reduced because of failures observed in the 1971 temblor.

—All quake-resisting frames in a building are to be required to be ductile—that is, to bend without breaking. Frames consist of metal or concrete.

—Better anchorage must be built for exterior appendages, such as veneer and outcropping, so they will not drop off.

—High-rise buildings (those more than 160 feet in height) will be required to have "fully dynamic seismic analyses," which means, Williams said, that plans for them must undergo a computer simulation of an earthquake to assure safe design.

Williams said that most high-rise structures have been undergoing this process, but not all.

—Essential facilities, such as hospitals, police and fire stations and communications centers, must be designed so that they are operational after a quake.

That means, Williams said, that:

—They must be able to withstand tremors "50% more effectively than any other buildings."

—All the equipment and service items must be properly anchored, so that they can absorb the movement and force of the quake.

Figure 1. United California Bank Building in Los Angeles. This 60-story high-rise steel frame building was designed to resist earthquake ground shaking by means of advanced dynamical methods that were developed by earthquake engineering research. The building is designed to withstand ground shaking produced by a magnitude 8+ earthquake on the San Andreas fault, as well as the ground shaking produced by smaller, closer earthquakes.

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ABSTRACT

Research on occurrence of earthquakes, on characteristics of earthquake ground motions, and on the motions of buildings during earthquakes, provided information for developing advanced methods of earthquake-resistant design of high-rise buildings. The tallest buildings in Los Angeles have been designed by these advanced methods. The ground and building motions recorded during the 9 February 1971 San Fernando, California, earthquake verified that the new methods of earthquake-resistant design were correct and, as a consequence, the Los Angeles Building Code has been revised to require that in the future high-rise buildings must be designed by these new dynamic methods. This is an example of how research can result in a practical payoff.

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1. Introduction.

The design of structures to withstand earthquake shaking is of great importance to public welfare and safety in the highly seismic regions of the United States. Even in the less seismic regions of the country, where earthquakes occur relatively infrequently, appropriate earthquake design of structures could be very important for public safety, for without it the community is highly vulnerable to an unexpected shock. When it was first recognized that earthquakes were an engineering problem, very little was known about the nature of the shaking to which structures are subjected during an earthquake, or the nature of the stresses and deformations that a structure undergoes during an earthquake. Therefore, the first requirement for developing the earthquake-resistant design requirements of a building code was to obtain accurate data about the frequency of occurrence of earthquakes, and their possible magnitudes, in order to assess the degree of hazard; and also to obtain a knowledge of the characteristics and severity of earthquake ground shaking and the nature of the building vibrations produced by it. This required knowledge has been, and continues to be, developed by research on earthquakes, on structural dynamics, and on soil mechanics; also by the development of appropriate instruments and by recording destructive ground shaking and building vibrations during earthquakes. Additional information is developed by the analysis of damage sustained during earthquakes. As such knowledge is amassed through research, and evaluated, it becomes incorporated in building codes and thus governs the design and construction of future buildings.

In the United States, the annual investment in construction is approximately $80 billion, and in the highly seismic regions of the country, approximately $10 billion per year is invested in construction. A change in the earthquake requirements in the building code which would increase the total construction cost in highly seismic regions by 5 percent would total $10 billion during the next 20 years. Similarly, a reduction of 5 percent would accumulate a savings of $10 billion during the following two decades. On the other hand, if the building code requirements are inadequate, a great earthquake striking a large city might cause $10 billion of losses and many deaths and injuries. It is clear that penalties for having inadequate building code requirements and the cost of having overly conservative building code requirements are so great that Governmental agencies tend to move very slowly in changing building codes.

2. Demonstrated Value of Good Earthquake Engineering.

The value of good earthquake engineering is illustrated by comparing the 9 February 1971 San Fernando, California earthquake with the 23 December 1972 Managua, Nicaragua earthquake. The magnitude 6.5 San Fernando earthquake struck the northern edge of the metropolitan Los Angeles area and shook with intense ground vibrations an area that included approximately 400,000 inhabitants.* The magnitude 6.2 Managua earthquake shook the entire city of 275,000 inhabitants with very severe ground shaking. The San Fernando earthquake caused 60 deaths, whereas the Managua earthquake caused in excess of 6000 deaths. This ratio of 100:1 in deaths is indicative of the quality of engineering and construction in the two cities. In Los Angeles, the building code and the practice of earthquake-resistant design were in an advanced state, whereas in Managua the building code and earthquake-resistant construction were generally in an undeveloped state. The total damage loss caused by the Managua earthquake has been estimated to be approximately equal to the annual Gross National Product of that country. It is clear that research in earthquake engineering has already made an important payoff in California. However, the San Fernando earthquake demonstrated that improvements are still required in the building code, and that research in earthquake engineering should be continued.

The development of earthquake-resistant design of high-rise buildings through research and the incorporation of modern requirements in the building code provide an informative example of how research in earthquake engineering pays off in increased public welfare and safety. This report describes the development of earthquake-resistant design for high-rise buildings in southern California.

3. History of Earthquake Codes.

Prior to 1933, there were no earthquake requirements in the building codes in the United States. Because of this, buildings in southern California constructed prior to 1933 are, in general, relatively hazardous during earthquakes as compared to buildings constructed after 1933. A few buildings were

*Engineering Features of the San Fernando, California Earthquake, P.C. Jennings, Editor, Earthquake Engineering Research Laboratory, June, 1971.

designed for earthquakes prior to 1933, notably the Southern California Edison Building in Los Angeles for which Professor R. R. Martel of the California Institute of Technology was consultant. However, in general, earthquake hazards did not figure in the thinking of building officials or design engineers. On March 10, 1933, at 5:45 p.m., the City of Long Beach, California was struck by a magnitude 6.2 earthquake. The loss of life (over 100) and the damage resulting from this earthquake quickly led to the incorporation of earthquake design requirements in the building code of Los Angeles and other southern California communities. (Appendix I) This initial requirement stated that all structures should be designed for a horizontal acceleration force equal to a specified percent of the weight of the building (this horizontal force is usually described as being a certain percent of gravity or %g).

The simple %g design requirement in the building code was not satisfactory as the magnitude of the design force was too low and also because the same force was specified for all buildings independent of their heights or their vibrational properties. At that time, zoning requirements limited building heights in Los Angeles to 150 feet, but even for buildings of that height, the constant %g requirement seemed inappropriate. A program of earthquake engineering research was carried out at the California Institute of Technology from 1930 to 1940 under the direction of R. R. Martel, aimed at investigating the earthquake behavior of multistory structures and developing appropriate building code requirements. Among the workers on this research program at the California Institute of Technology were Maurice A. Biot, Merit P. White, George W. Housner, and others. The research was sponsored by the Los Angeles County Building Department. A portion of the research was carried out at Stanford University by Lydik S. Jacobsen, Nicolas J. Hoff, and others. This research developed a better understanding of the vibrations of structures during earthquakes. One of the most significant results of the research was the development of the Earthquake Response Spectrum and the spectrum techniques now widely used for earthquake-resistant design of structures. The results of this research were incorporated in new building code requirements. Instead of a constant %g, the earthquake design forces were specified as a function of the height of the building thus, in effect, making the design requirements dependent on the natural period of vibration of the structures. This was the first earthquake code based on dynamics.* These new requirements, incorporated in the Los Angeles Building Code and in the Uniform Building Code, were formulated only for buildings up to 13 stories in height (150 feet). At this time, San Francisco had no earthquake requirements in the building code, and permitted the construction of buildings exceeding 13 stories in height. In the mid-1950's, the earthquake design requirements in the code were revised so as to apply to high-rise buildings, and to make the distribution of horizontal design forces and the bending moments more obvious, and both San Francisco,† Los Angeles,‡ and the Uniform Building Code adopted these revised code requirements. Since that time, numerous modifications have been made to the code. The usual process for making these changes is for the Structural Engineers Association of California to draw up recommended changes for the building code which are submitted to the building departments of Los Angeles and San Francisco, and are also submitted to the International Conference of Building Officials, which is responsible for the Uniform Building Code. These changes are usually adopted and appear in all of the building codes. Sometimes, however, code changes are initiated by the building departments themselves.

*These requirements were adopted by the City of Los Angeles in 1943; and by the Uniform Building Code in 1946.

† 1954

‡ 1957

4. Pre-1971 Building Code Requirements for High-Rise Buildings.

The earthquake design requirements of the Los Angeles Building Code prior to 1971 are given in Appendix II. Essentially, so far as high-rise buildings were concerned, the code specified the distribution of horizontal acceleration forces over the height of the building and specified how the horizontal shear forces and the bending moments were to be calculated. This method of specifying design forces left much to be desired for it essentially treats a dynamics problem as if it were a statics problem.

These simplified code requirements aimed at providing structures with adequate earthquake resistance, but the formulation of the code, in the form of a statics problem, had certain deficiencies. The magnitude of the acceleration forces specified by the code was much less than the acceleration forces actually experienced by buildings during strong earthquakes. This was established by research which showed that the ground shaking recorded during strong earthquakes would actually produce relatively large acceleration forces in the upper parts of high-rise buildings. On the other hand, the allowable design stresses permitted by the code had been determined on the basis of static tests and underestimated the true dynamic resistance of the buildings. These two factors tended to counterbalance each other in a way which might, or might not, lead to a satisfactory design. Also, it was not possible to specify both the correct dynamic shear forces and the correct bending moments in the building by means of the equivalent static acceleration force used by the code.

5. Design of High-Rise Buildings in Los Angeles.

In the early 1960's the height limit was removed from the Los Angeles Building Code. Buildings of any height were permitted, but the restriction on the volume of a building remained. The volume of a high-rise building could not exceed the volume of a 150 ft high building which used all of the building area of the lot, so control of the population density was retained. Following the removal of the restrictions on the height of buildings in Los Angeles, a number of high-rise buildings were constructed. The large financial investment represented by a high-rise building led the owners of these buildings to specify that special consideration should be given to earthquake design. Professors P. C. Jennings and G. W. Housner of the California Institute of Technology were consultants on the design of most of the high-rise buildings in Los Angeles. These included the Union Bank Building, 42 stories; the Atlantic-Richfield Twin Towers, 52 stories; the United California Bank Building, 60 stories; the Security-Pacific Bank Building, 60 stories. All of the latest research results were used in specifying the earthquake design criteria for these structures. (Most of this research was sponsored by the National Science Foundation). The general procedure used for the earthquake design of these high-rise buildings was as follows: (a) The locations of earthquakes likely to occur in southern California, and their magnitudes, were estimated on the basis of the seismic history and the tectonic processes operating in southern California. Ground motions corresponding to these design earthquakes were synthesized and the response of the building was computed for each of these ground motions. (See report: Simulated Earthquake Motions, by P. C. Jennings, G. W. Housner, and N. C. Tsai, Earthquake Engineering Research Laboratory, California Institute of Technology, April, 1968). The maximum earthquake shear forces and bending moments acting on the building during each of these earthquake ground motions was calculated by means of a digital computer, and the final design of the building was made so that the structure would be safe in the event that any of these ground motions were to occur. This approach differed markedly from the

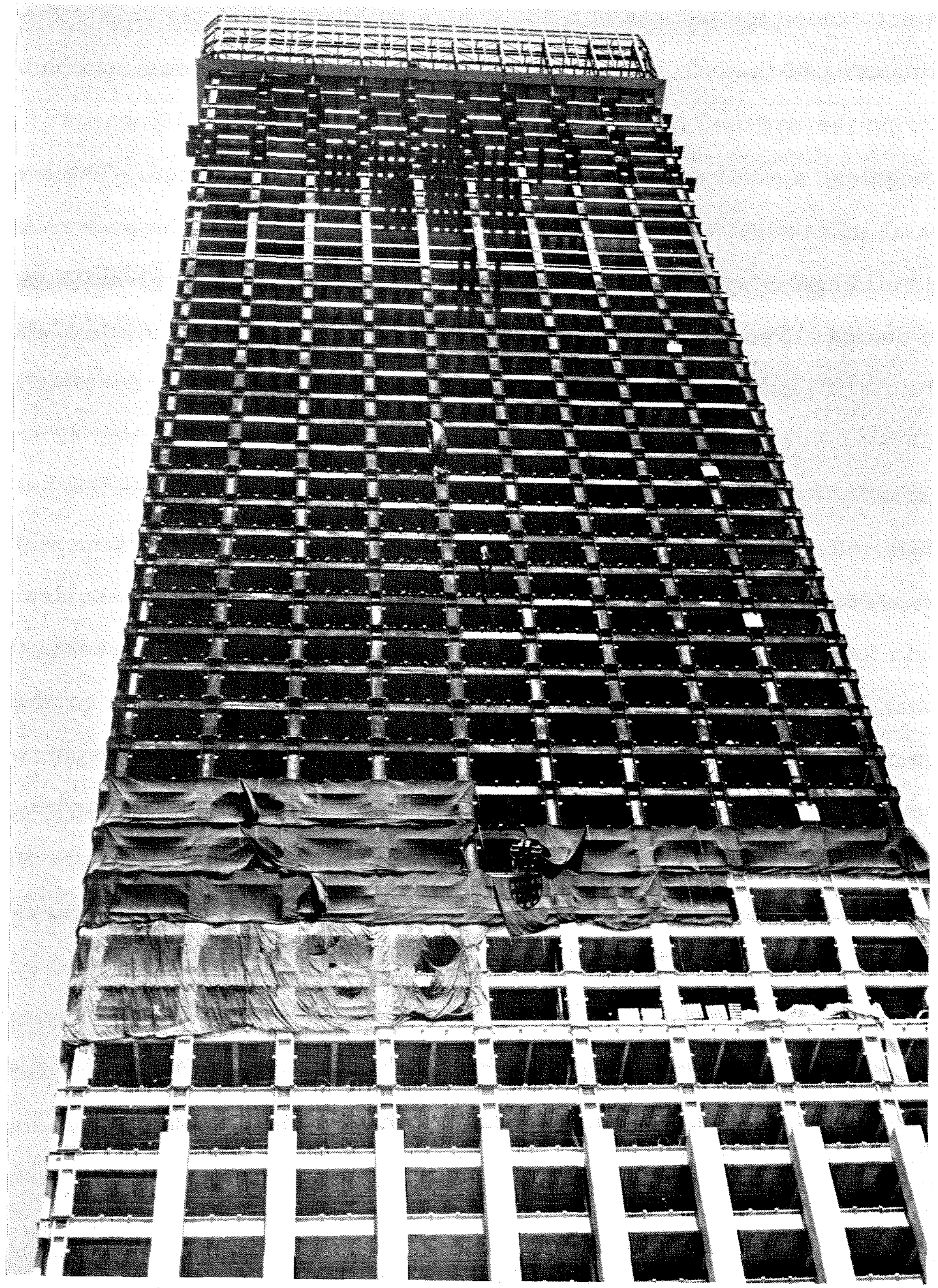


Figure 2. The Security-Pacific National Bank Building under construction. This 60-story high-rise building in Los Angeles is designed to withstand the strongest shaking that might be generated by earthquakes on any of the faults in southern California.

Figure 3. Twin 52-story Atlantic-Richfield buildings under construction. This photograph was taken immediately following the 9 February 1971 San Fernando, California earthquake. The buildings went through the earthquake without any evidence of damage.

Figure 4. Union Bank Building in Los Angeles, California. This 42-story, steel frame structure was the first high-rise building to be erected in Los Angeles. It safely survived the San Fernando earthquake.

earthquake requirements specified in the building code and, moreover, was free of the defects that existed in the code requirements. The validity of this new approach depended upon the ability to make reasonable assessments of the ground motions likely to occur at the site, the ability to make accurate digital computations of the response of the building given the ground shaking at the base, and the ability to make a reasonable judgment as to the real strength of the structural members and connections. Although the research workers were confident of their ability to do these things, engineers and building officials were not entirely convinced. When the 9 February 1971 San Fernando earthquake occurred, recordings were obtained of ground shaking and of building vibrations in many structures. These recordings, and analyses of them, demonstrated that the new approach to the design of high-rise buildings was indeed valid. This evidence was sufficiently convincing so that the Los Angeles Building Code was revised to specify that high-rise buildings should be designed along the lines of this new dynamic approach that had been employed for the design of the foregoing buildings (Appendix III).

6. Design Earthquakes Used for High-Rise Buildings.

The geology of southern California has been well studied, and there is no disagreement about major active faults on which large earthquakes can be expected to occur. The most prominent fault is the San Andreas fault which, at its closest point, is approximately 35 miles from the center of Los Angeles. There is general agreement among seismologists that a magnitude 8+ earthquake may be generated on this fault, and that it is probable that such an earthquake will occur within the next 100 years. Even though the causative fault is 35 miles from the center of the city, the ground motion generated by the earthquake can be expected to put a severe test to a high-rise building. The fundamental period of vibration of a high-rise building is approximately 0.1 sec for each story, so that a 50-story building can be expected to have a fundamental period of about 5 secs. The long-period components (approximately 5 secs) in the seismic waves generated by a magnitude 8+ earthquake will not attenuate with distance as rapidly as do shorter wavelengths and, therefore, the ground motion affecting a 50-story building that is 35 miles from the causative fault will still be very severe. Other, shorter faults in southern California can be expected to generate earthquakes of smaller magnitudes than 8, but if they are closer to the building site than 35 miles their effects may still be as severe, or even more severe, than the effects of a large earthquake on the San Andreas fault. For example, a magnitude 7.0 might be generated on the Newport-Inglewood fault at a distance of 15 miles from the site, and this might produce building vibrations more severe than the earthquake on the San Andreas fault. For the design of the high-rise buildings, different design earthquakes were postulated, ranging from a magnitude 8+ on the San Andreas fault to a magnitude 5.5 within one mile of the site. (Professor C. R. Allen, California Institute of Technology was consultant on fault activity and expected earthquakes)

One of the uncertainties involved in specifying earthquake design requirements is that for any particular fault, the expectation of an earthquake varies inversely as a magnitude of the shock. For example, an earthquake of magnitude 6 or greater might be expected during the next 100 years with a relatively high probability, whereas a magnitude 7, or greater, earthquake on the same fault might be expected during the next 100 years with a very low probability. The probability of experiencing the design earthquake must be taken into account by establishing the overall design process. For example, if the design earthquake has a 50% probability of occurring within the next 50 years, the allowable stresses and strains used in the design should be different than if the design earthquake has only a 50% probability of occurring within the next 500 years. In this latter case, the design stresses might be relatively high and large strains might be permitted. In this way, a magnitude 6.5 earthquake which has a high degree of probability of occurring during the life of the structure may lead to the same design as a magnitude 7.5 earthquake which has only a low probability of occurring during the life of the building.

7. Earthquake Ground Motions for Design of High-Rise Buildings.

The most desirable method of establishing design ground motions would be to use actual earthquake ground motions recorded under similar conditions. For example, if ground motion had been recorded in the center of Los Angeles for a magnitude 8+ earthquake on the nearby San Andreas fault (the last occurring in 1857) this could then be used for making the design of a high-rise building. Actually, ground motions have never been recorded within 50 miles of the causative fault of a magnitude 8+ earthquake, and because of this, the ground motion in the center of Los Angeles for such an earthquake must be extrapolated from the various earthquake records which have been recorded. From earthquake ground motions that have been recorded at various distances from the faults that have generated earthquakes of various magnitudes*, reasonable estimates can be made of the duration of strong ground accelerations to be expected, the frequency content of the accelerogram, and the

*Some of the ground motions were recorded by strong-motion accelerographs that had been installed as part of National Science Foundation sponsored research programs.

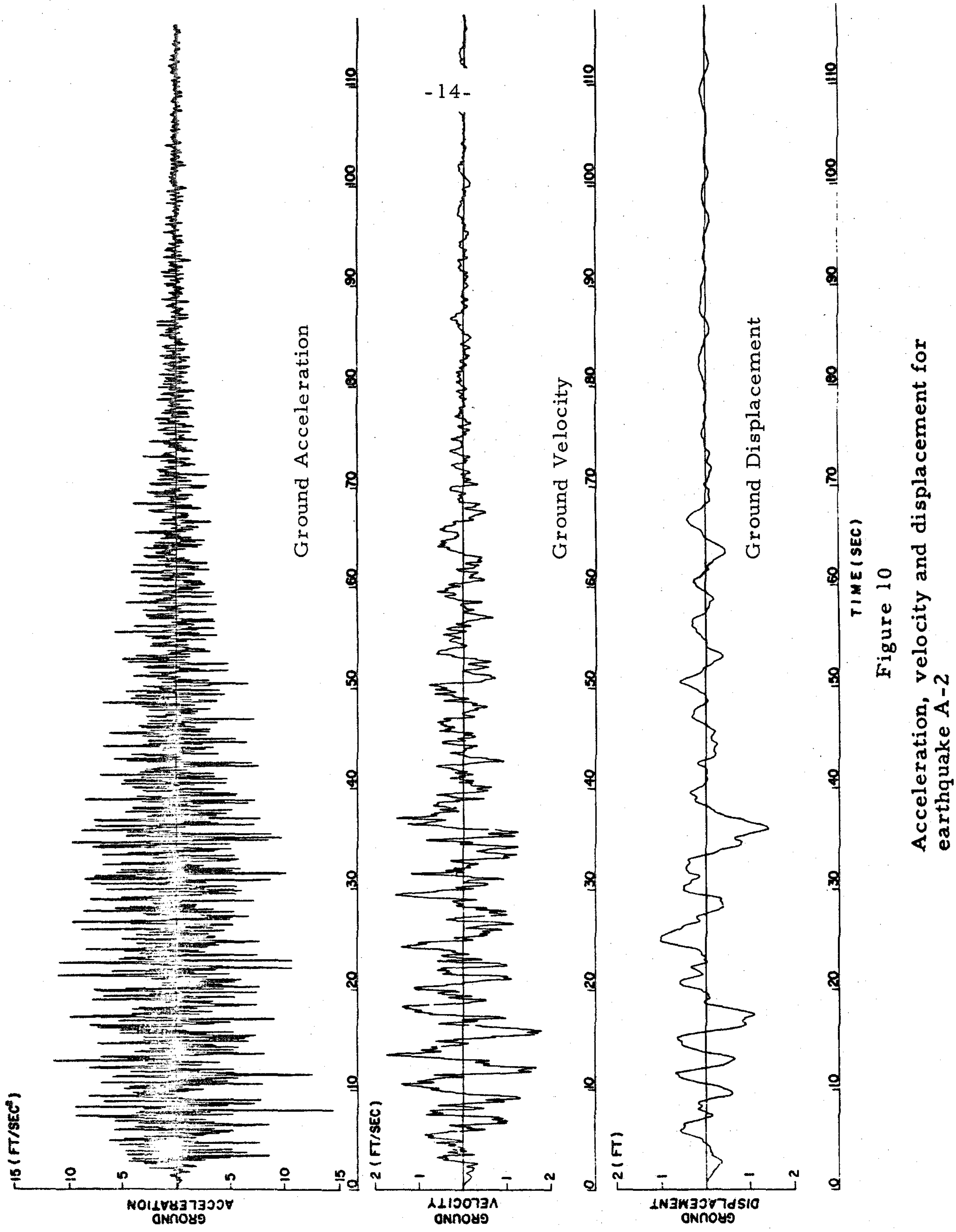


Figure 5. Synthetic accelerogram representing the ground motion close to the causative fault of a magnitude 8.5 earthquake on the San Andreas fault. This ground motion, when multiplied by the factor 0.67 represents the ground motion in central Los Angeles corresponding to a magnitude 8.5 earthquake on the San Andreas fault, 34 miles to the northeast. This earthquake ground motion is designated as Earthquake A-2.

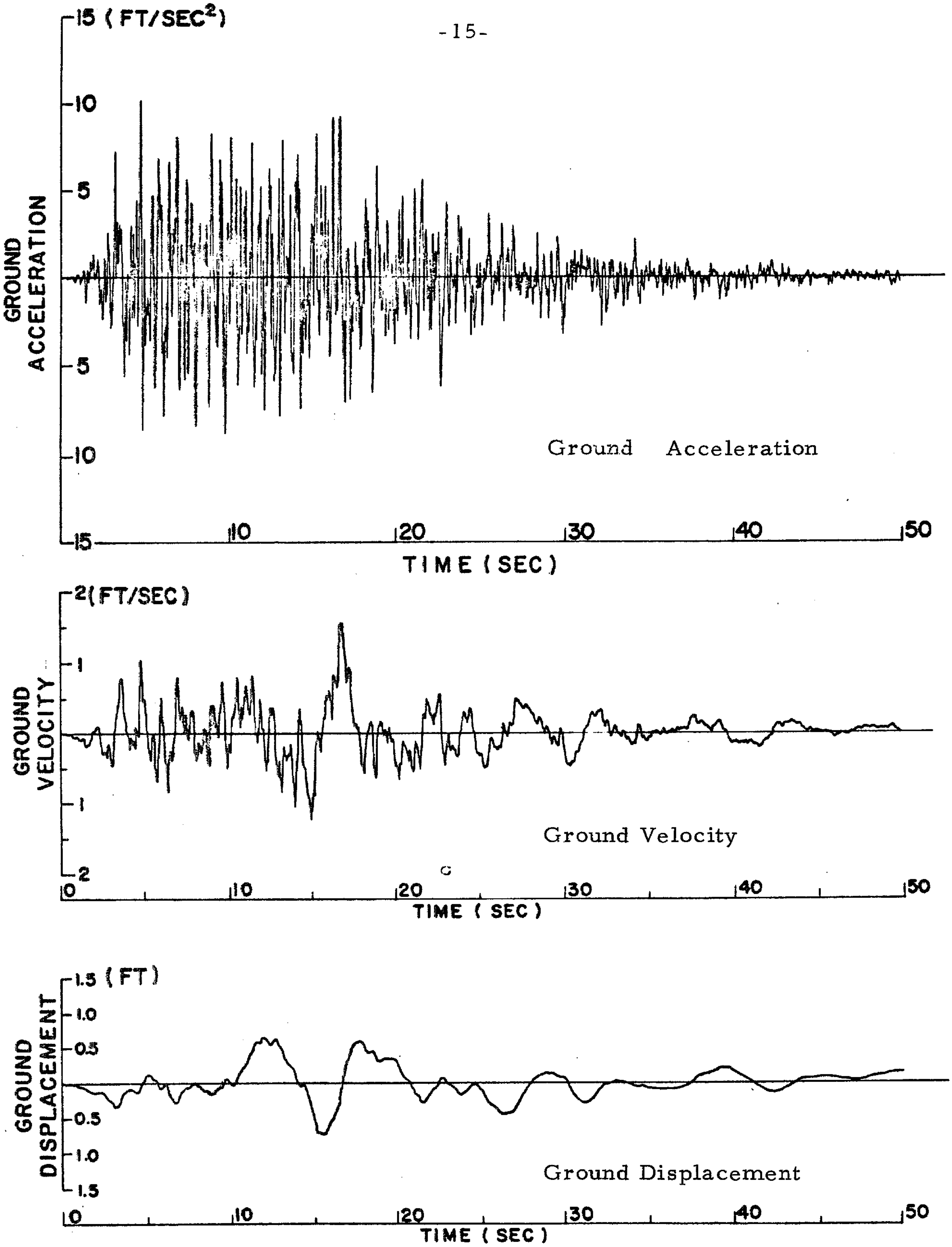


Figure 6. Synthetic ground motion representing the motion in central Los Angeles caused by a magnitude 7 earthquake on the Newport-Inglewood fault, ten miles distant. This ground motion is designated as Earthquake B-2.

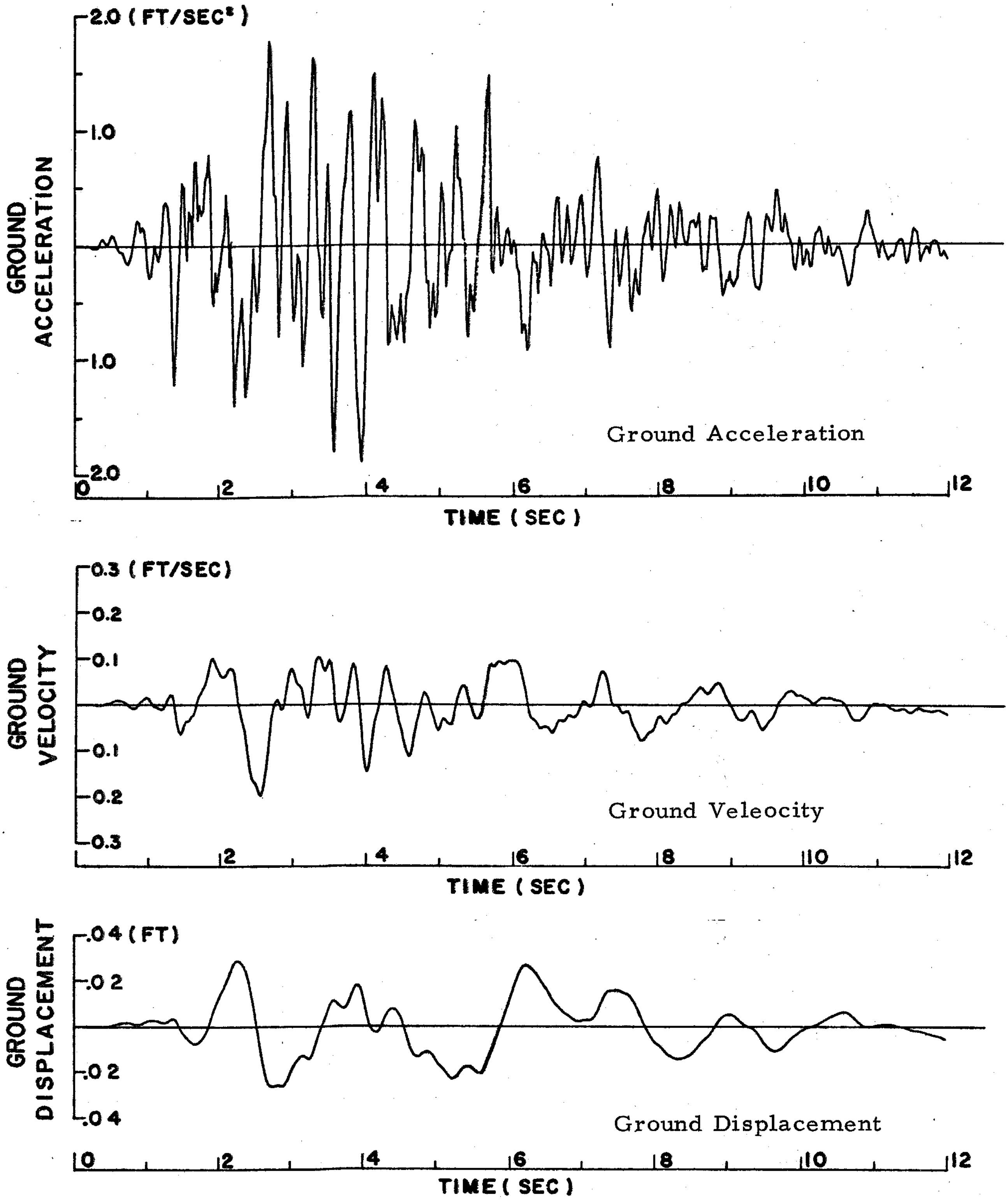


Figure 7. Synthetic ground motion representing the ground acceleration generated by a magnitude 5.5 earthquake at a distance of approximately 5 miles.

general energy content. With this knowledge, it is possible to generate simulated earthquake accelerograms corresponding to earthquakes of various magnitudes at various distances. By means of this technique, appropriate synthetic ground accelerations were prepared for each design earthquake to be used for analyzing the high-rise buildings. Samples of these are shown in Figures 5 to 12.

8. Calculated Response of Buildings.

For each of the design earthquakes, the dynamic response of the high-rise building was calculated by means of a digital computer. The customary procedure was to calculate the six to ten lowest modes of vibration, that is, their natural periods of vibration and their mode shapes. The time-history response of each mode of vibration to the ground motion was then calculated. The summation of the responses of all the modes of vibration then gave the building response. The building response was scanned to determine the maximum interfloor shear force at the various story heights during the earthquake, the maximum overturning moments at the various story heights, the maximum displacements of the floors, and the maximum acceleration at each floor. The foregoing quantities were determined for each of the design earthquakes and an appropriate design of the structure was then made. See Figures 8 through 16.

9. San Fernando Earthquake Used as a Check.

At the time of the San Fernando earthquake, all new buildings over ten stories in height in Los Angeles had been outfitted with recording accelerographs, as required by the Los Angeles Building Code. These instruments were installed, one in the basement of the building, another on the top floor of the building, and a third at midheight*. The records provided by these

*When the requirement for instrumenting high-rise buildings was incorporated in the Los Angeles Building Code, the California Institute of Technology was asked to specify the desired characteristics of the recording instruments. Caltech also agreed to check the various types of instruments prepared for this use and to approve those that were satisfactory. The knowledge for doing this was, in large part, obtained from research projects sponsored by the National Science Foundation.

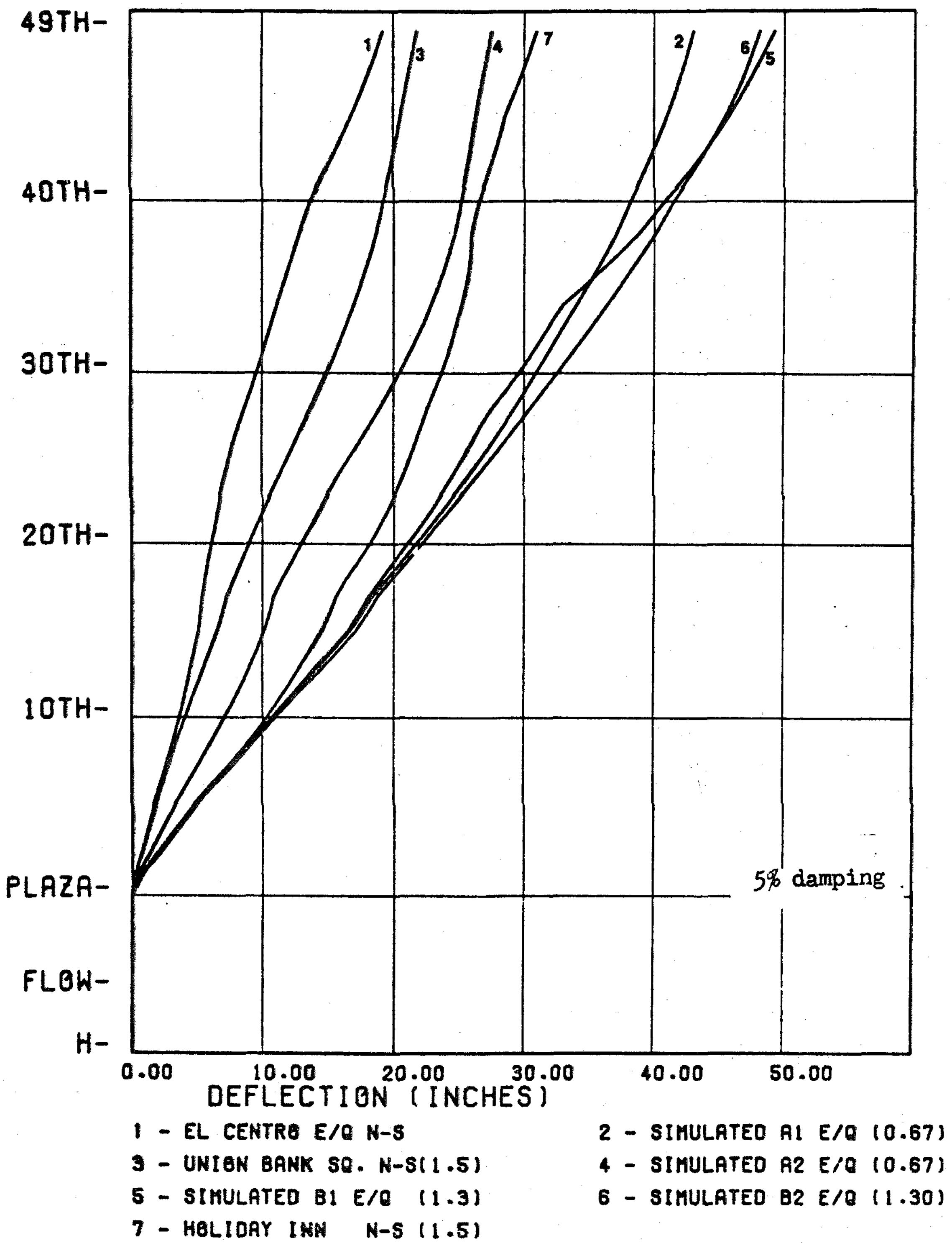


Figure 8. Maximum building deflections computed for seven different earthquake ground motions for the design of the Security-Pacific National Bank Building. Albert C. Martin & Associates were architect-engineers for the building.

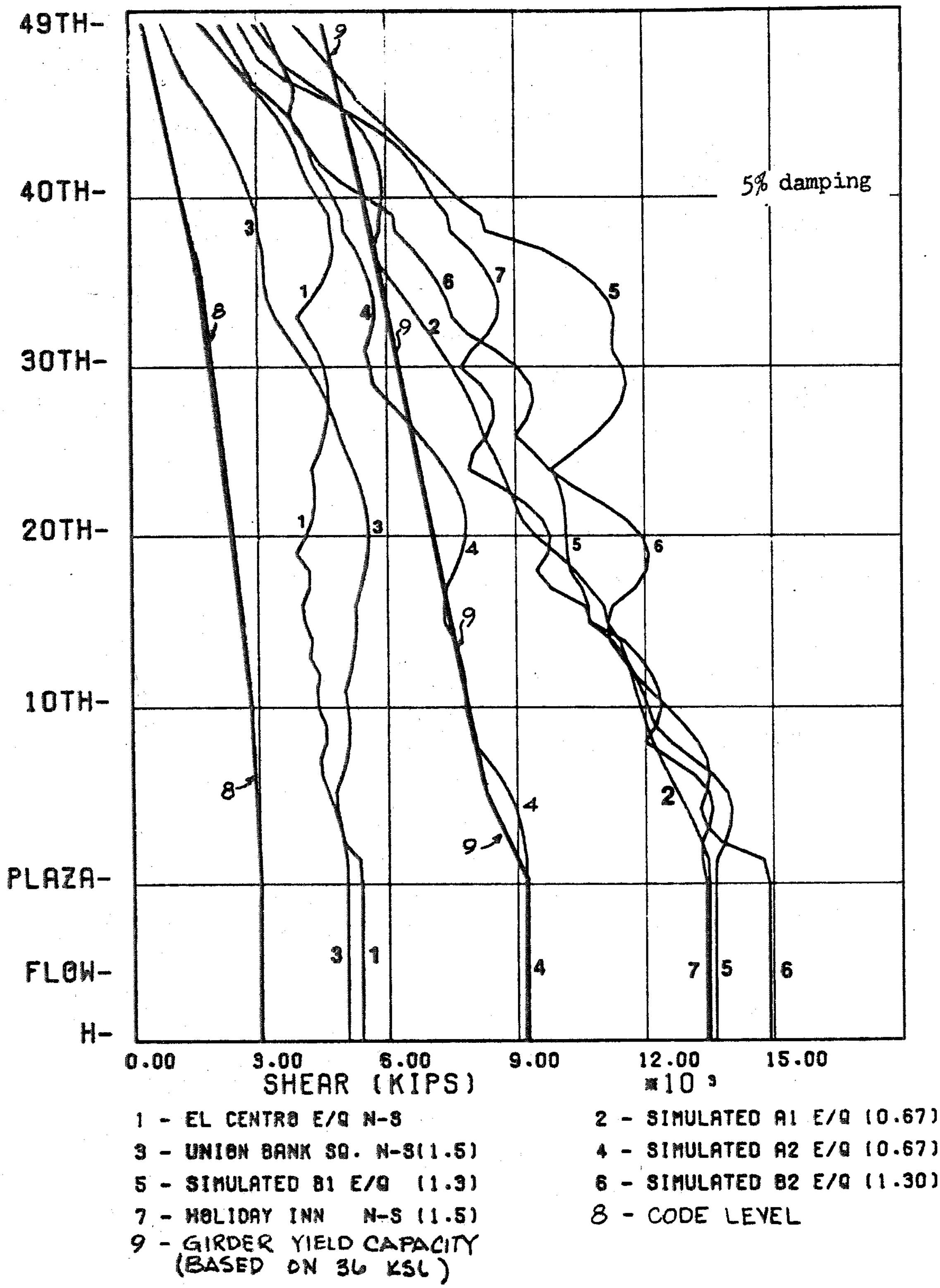


Figure 9. Maximum shear force in millions of pounds calculated for seven different earthquake ground motions for the design of the Security-Pacific National Bank building. A. C. Martin & Associates were architect-engineer for the building.

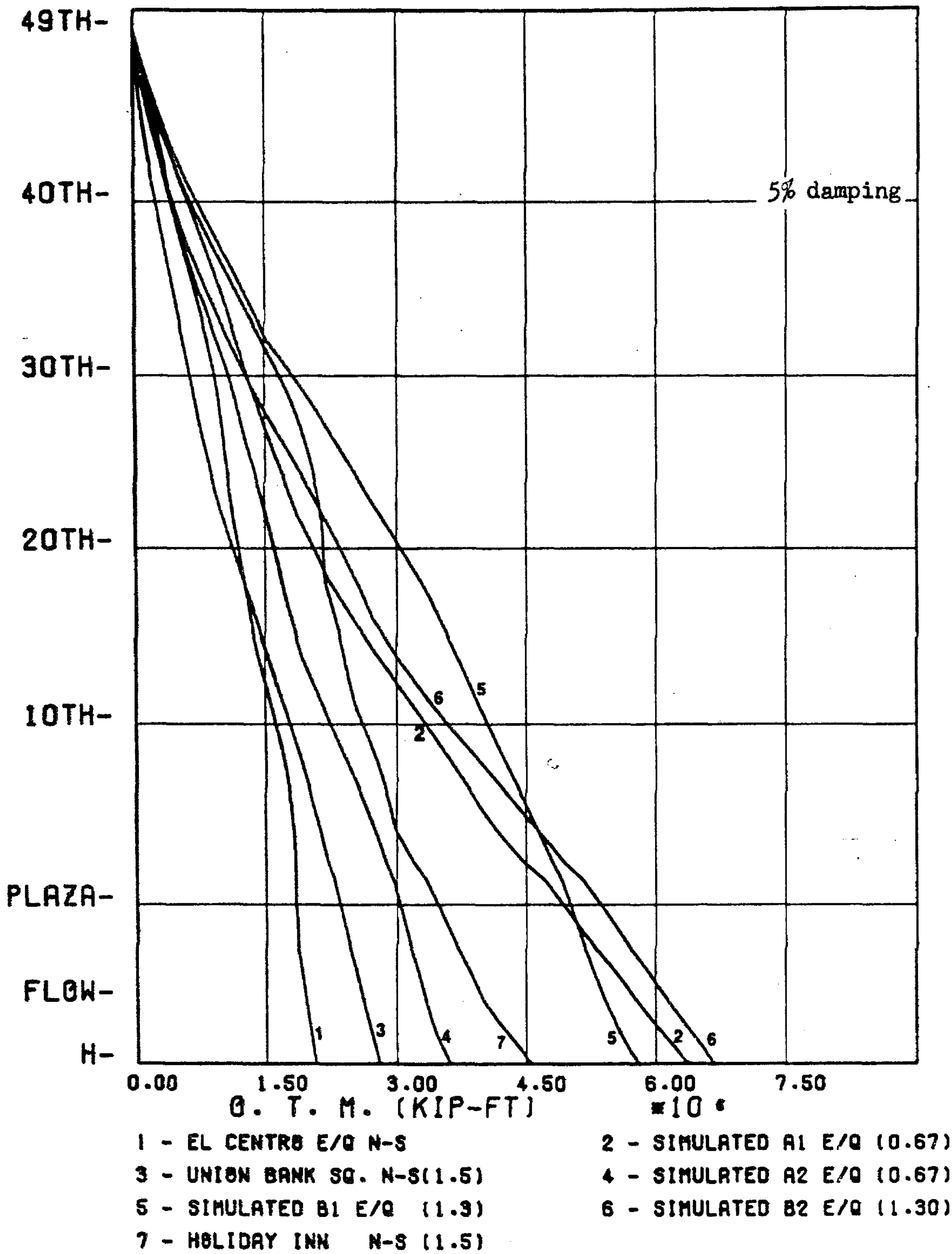


Figure 10. Maximum overturning moments in billions of ft/lbs calculated for seven different earthquake ground motions for the design of the Security-Pacific National Bank Building. Albert C. Martin & Associates were the architect-engineer for the building.

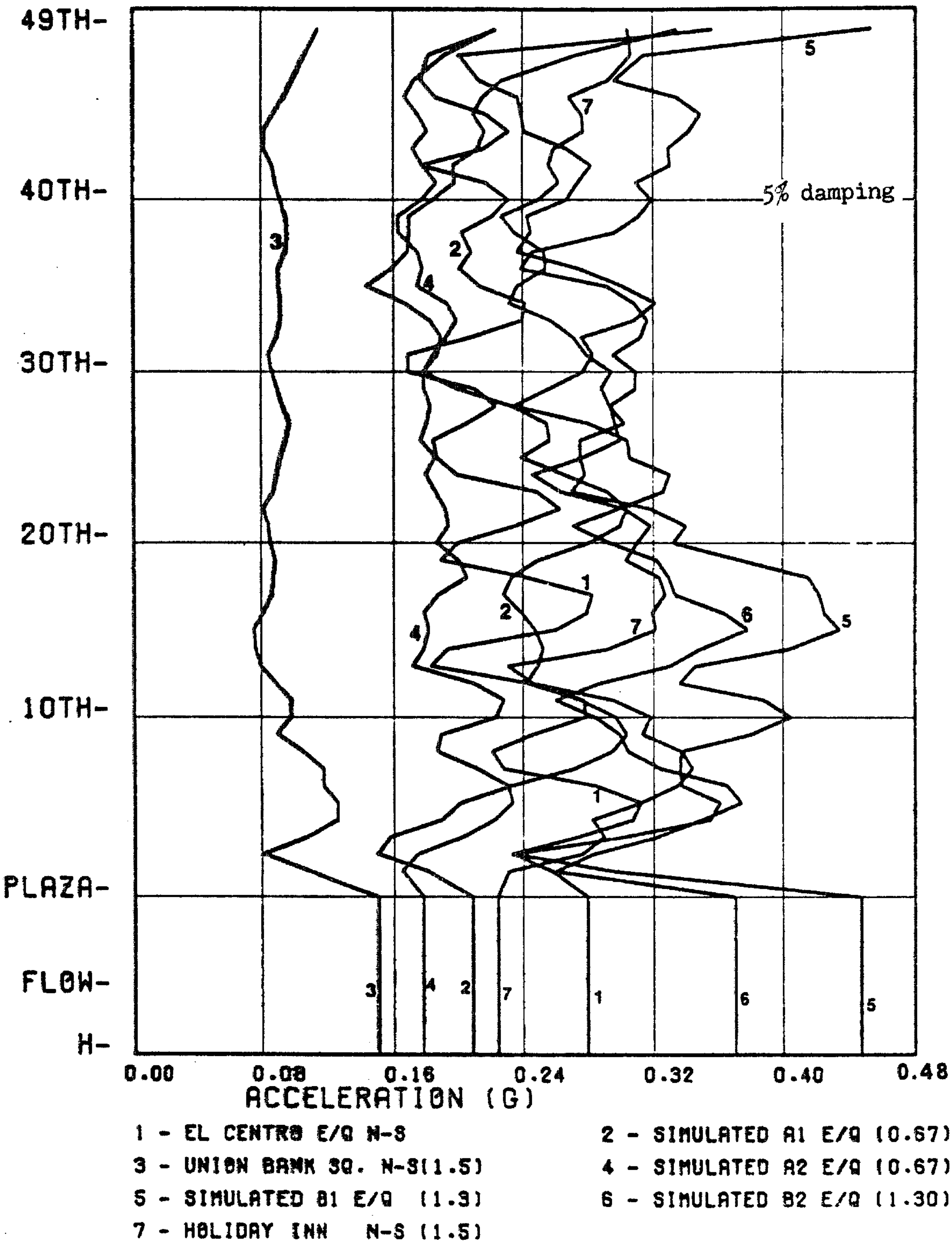


Figure 11. Maximum acceleration at each floor level computed for seven different ground motions for the design of the Security-Pacific National Bank Building. A. C. Martin & Associates were architect-engineer for the building.

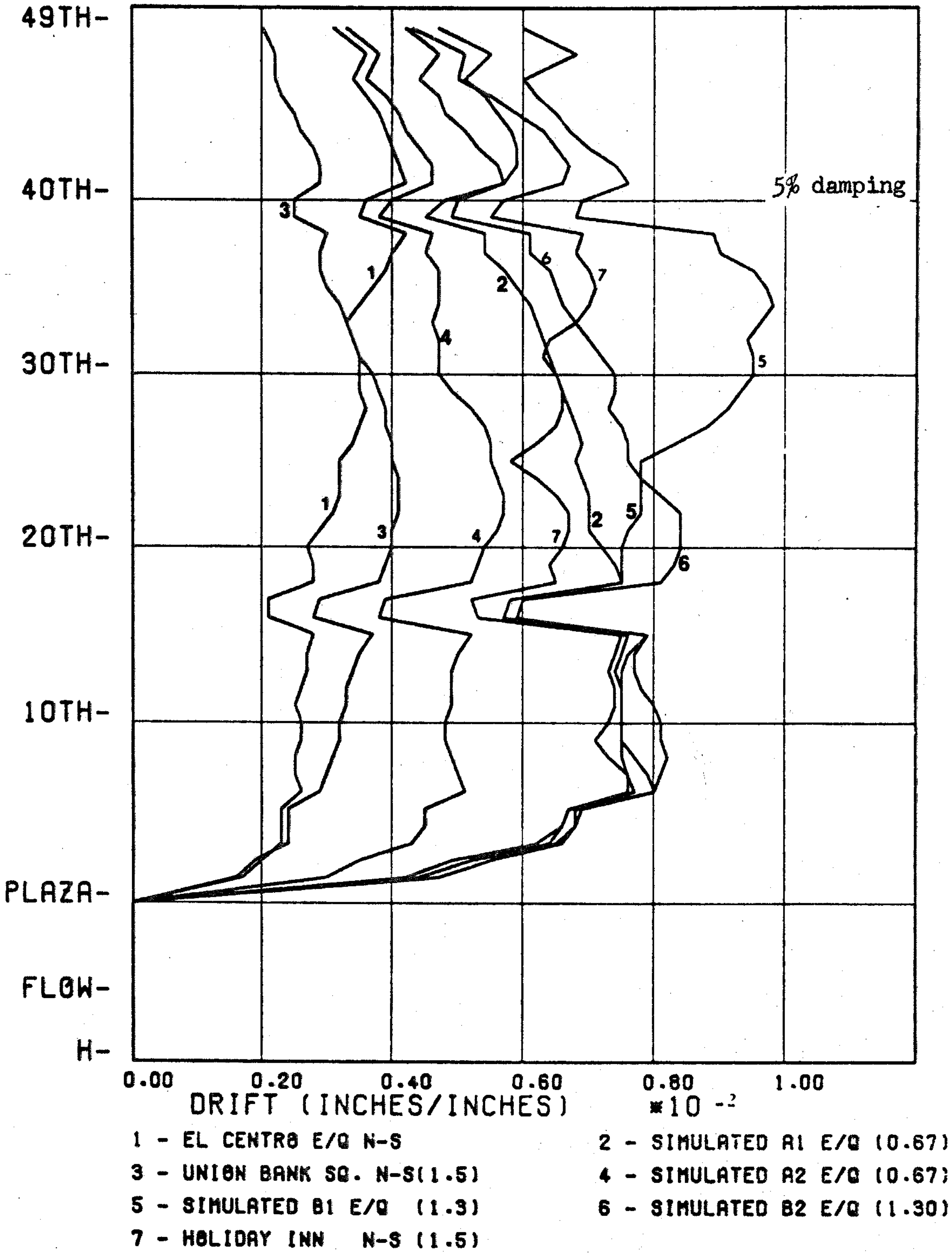


Figure 12. Maximum inter-story deflection calculated for seven different earthquake ground motions for the design of the Security-Pacific National Bank Building. A. C. Martin & Associates were architect-engineer for the building.

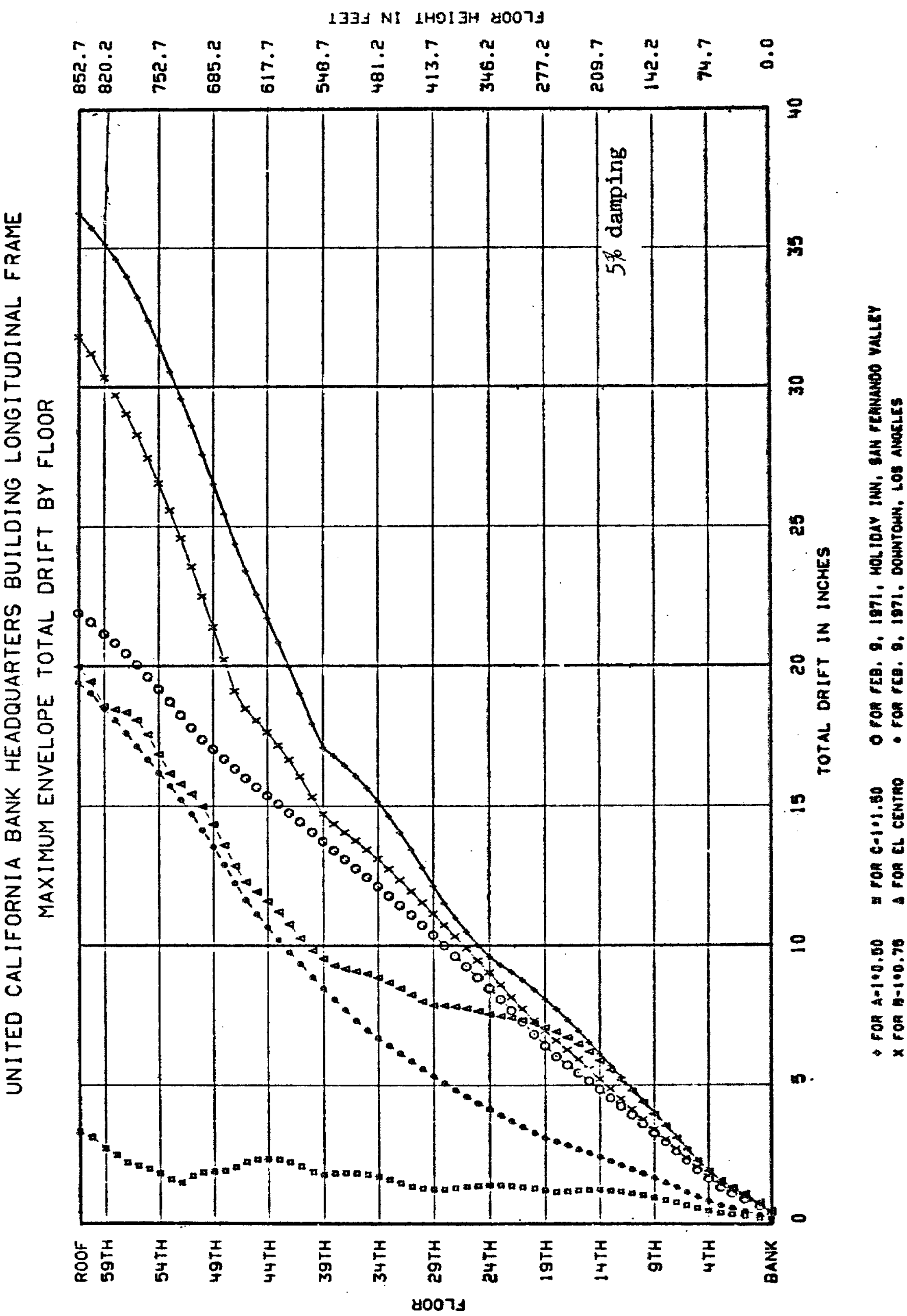


Figure 13. Maximum building displacement calculated for six different earthquake ground motions for the design of the United California Bank Building. Erkel and Greenfield did the engineering design of the building.

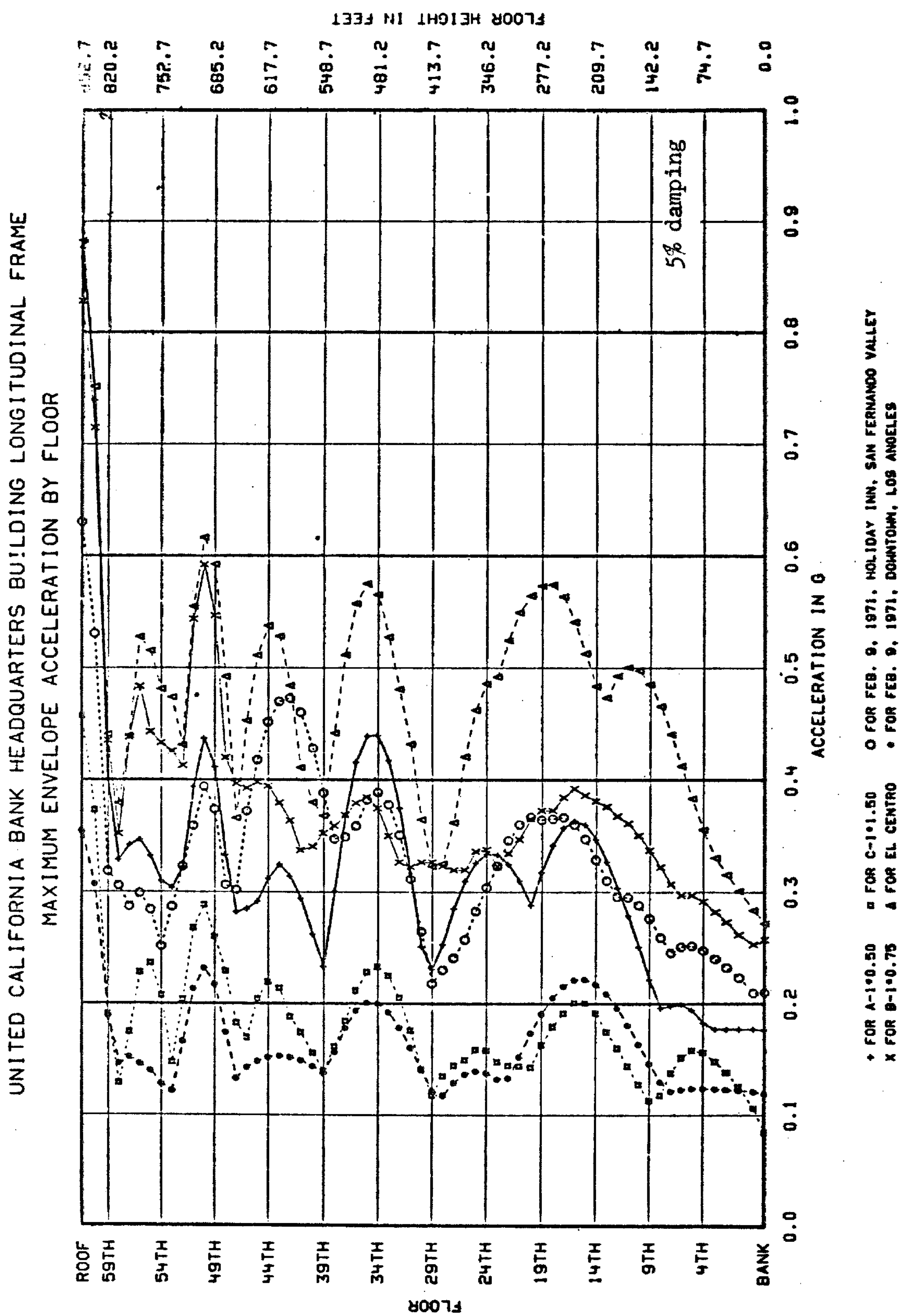


Figure 14. Maximum acceleration of each floor calculated for six different earthquake ground motions for the design of the United California Bank Building. Erkel and Greenfield, Structural Engineers.

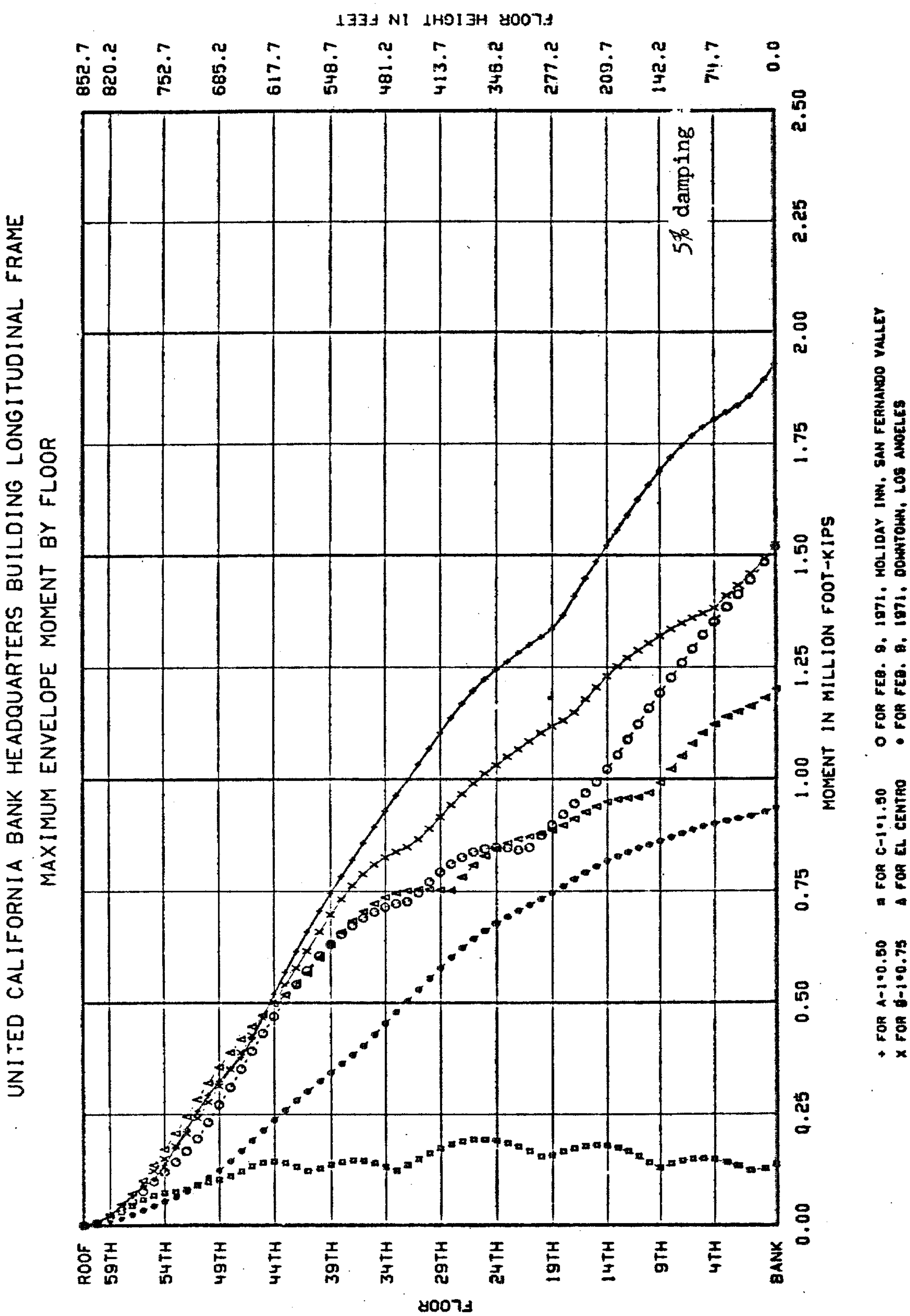


Figure 15. Maximum overturning moments in billions of ft/lbs calculated for six different earthquake ground motions for the design of the United California Bank Building. Erkel and Greenfield, Structural Engineers.

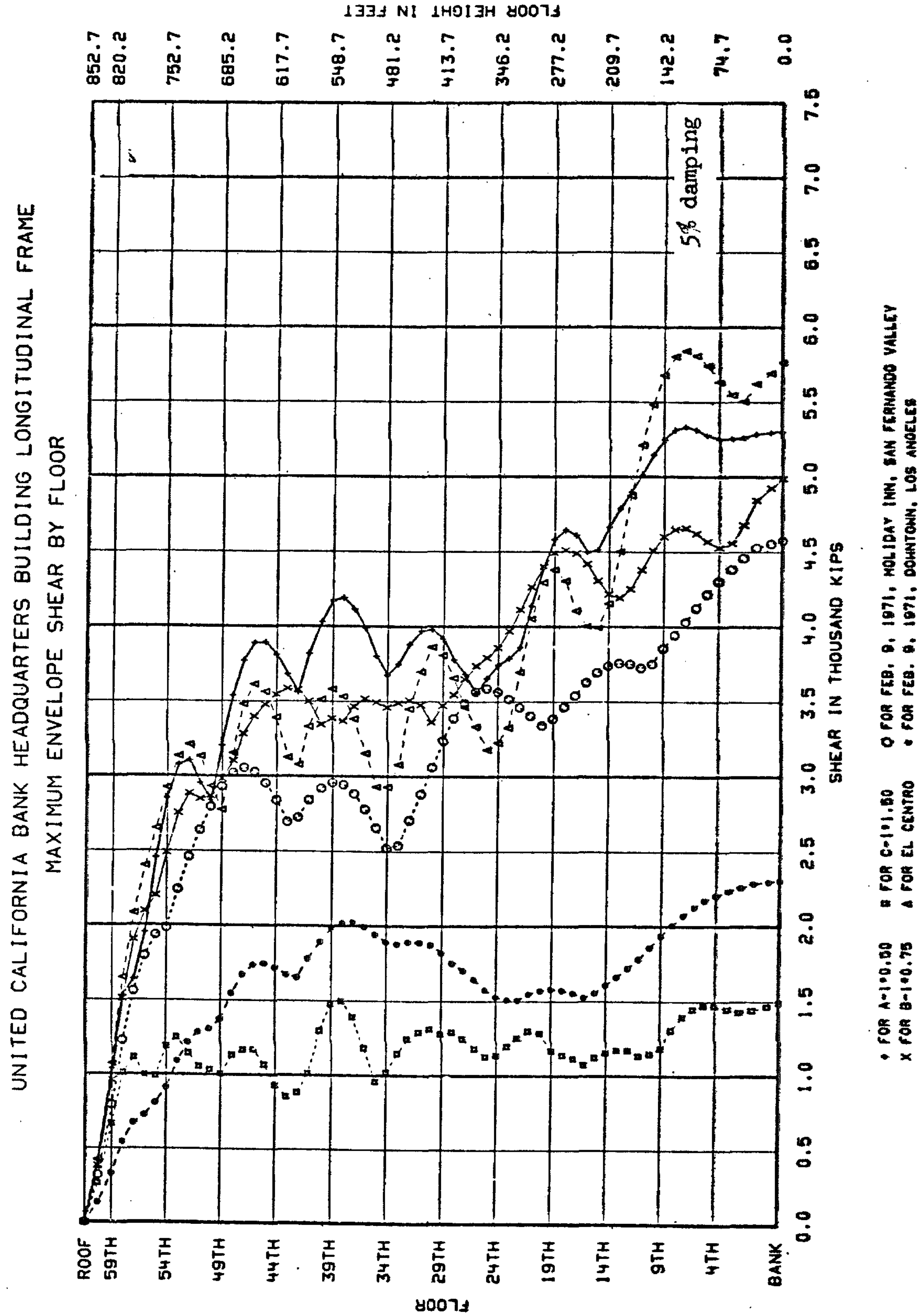


Figure 16. Maximum shear forces in millions of pounds calculated for six different earthquake ground motions for the design of the United California Bank Building. Erkel and Greenfield, Structural Engineers.

instruments gave a complete description of the motion of their points of attachment to the buildings during the earthquake.* This was the first time that such strong shaking had been recorded in multistory buildings. The nature of the motion recorded in the basements of the buildings was consistent with the motion that had been postulated for the design of the high-rise buildings. In addition, the records obtained in the upper parts of the buildings provided the opportunity to check the calculations of building responses. Taking the recorded basement motion as an input, it was now possible to calculate the response of the building and to see how well it agreed with the motions that actually had been recorded during the earthquake. Very good agreement was obtained in this way.

Such a check was made on the multistory steel-frame Administration Building at the Caltech Jet Propulsion Laboratory in Pasadena. The laboratory is located near the northern edge of the city at the base of the San Gabriel Mountains, approximately 15 miles east of the center of the San Fernando earthquake. After the San Fernando earthquake, while excavating for a bridge foundation on the Jet Propultion Laboratory grounds, an active fault was exposed trending downward to the north at about 45°. This fault had granite rock above and gravel below, thus indicating that the granitic San Gabriel Mountains had been overthrusting the alluvium of the valley. Explorations that had been made where the San Fernando fault had intersected the surface of the ground showed similar overthrusting of granite above alluvium, and it is clear that the Jet Propulsion Laboratory fault is just an extension of the same fault system on which the San Fernando earthquake was generated. This, then, raised the question, what would be the behavior of the Jet Propulsion Laboratory buildings in the event that there should be a repetition of the San Fernando earthquake, but located 15 miles to the east, so that the Jet Propulsion Laboratory would

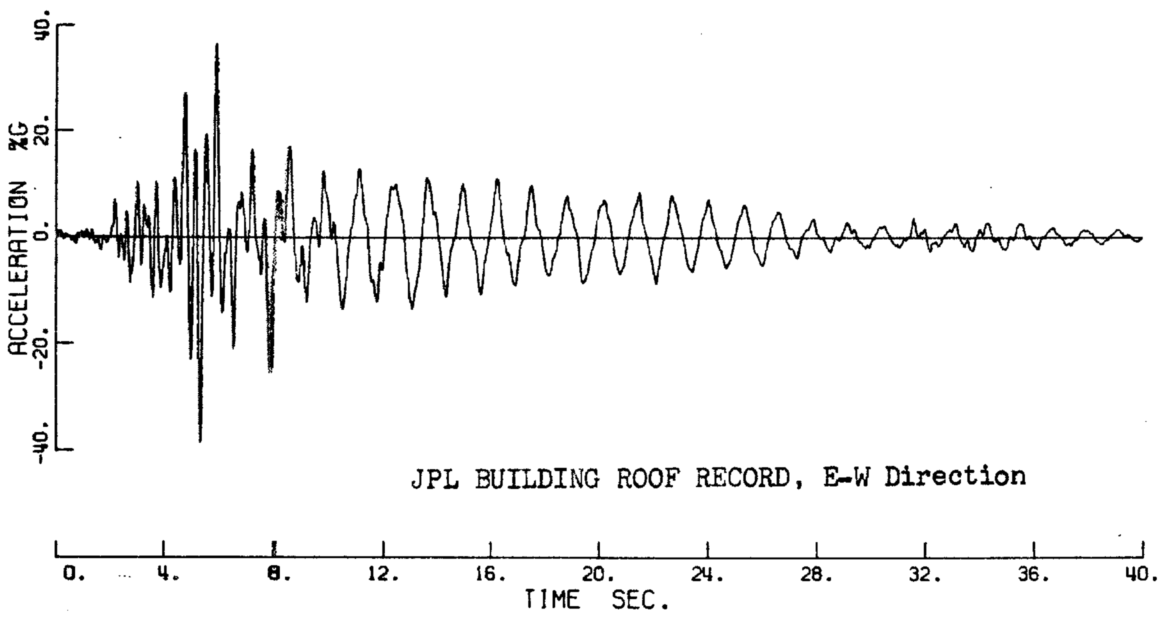
*Strong Motion Earthquake Accelerograms, Digitized and Plotted Data, vols. Ic, d, e, f, Earthquake Engineering Research Laboratory, July, 1971.

be at the center of the earthquake? The first step in answering this question was to analyze the response of the JPL Administration Building to the motions generated by the San Fernando earthquake. Fortunately, as part of an NSF sponsored research project, an accelerograph had been installed in the basement of the building, and another on the roof in anticipation of an earthquake. The basement instrument recorded a peak acceleration of 20%g and the instrument on the roof recorded a peak acceleration of 40%g. Ground and building motions are shown in Figures 17 through 20*. Comparison of the building motion computed from the recorded basement motion with that actually recorded at the top of the building during the earthquake is shown in Figures 9, 10, and 11. It is seen that there is very good agreement between the computed building motion and the recorded building motion. This is convincing evidence that the vibratory response of a building can be calculated accurately when the motion of the base of a building is known. Similar checks were made for various multistory buildings, and equally good results were obtained.

10. New Code Requirements for High-Rise Buildings.

The Los Angeles Building Department had approved the use of the dynamic analysis and design for high-rise buildings before the San Fernando earthquake, but the convincing evidence provided by the earthquake led to a revision of the building code which makes mandatory a dynamic design for future high-rise buildings. The pre-1971 earthquake requirements of the building code are given in Appendix II, and the revisions of the building code resulting from the San Fernando earthquake are given in Appendix III. The requirements for high-

*The analysis of this building motion is described in the report "Analysis of the Earthquake Response of a 9-Story Steel-Frame Building During the San Fernando Earthquake," John H. Wood, Earthquake Engineering Research Laboratory, California Institute of Technology, October, 1972.



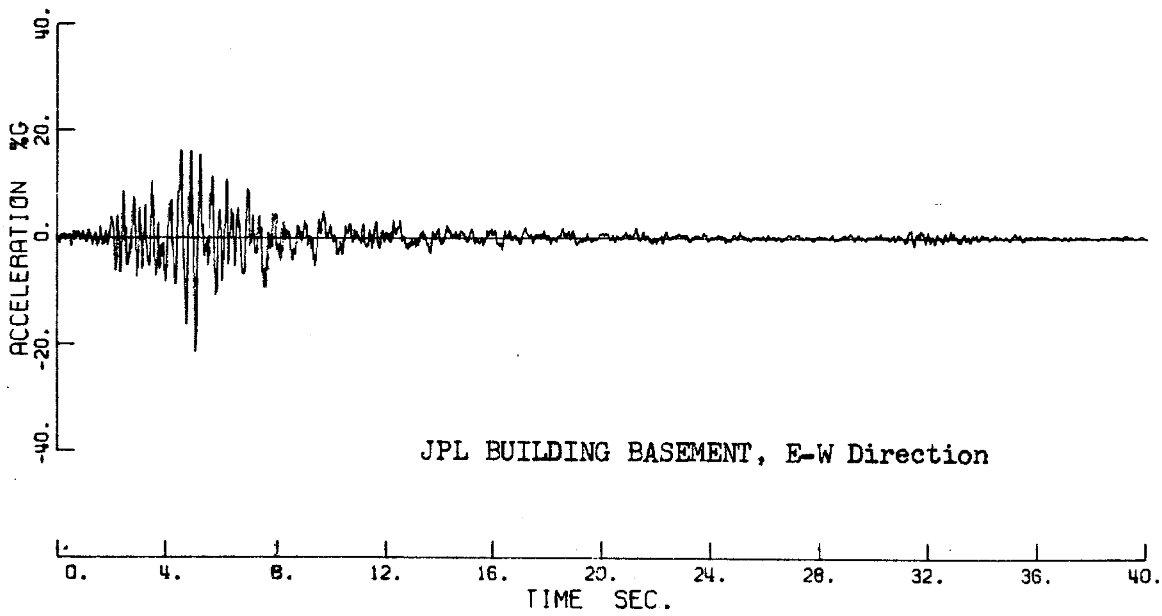
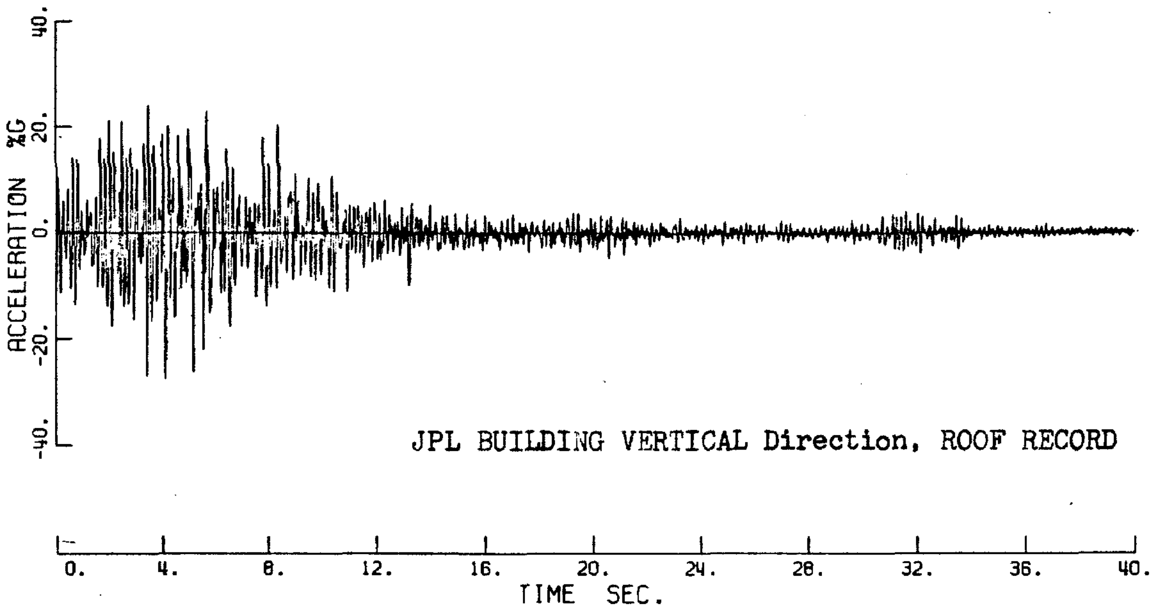


Figure 17. Earthquake motions recorded during the 9 February 1971 San Fernando, California earthquake in the basement and on the roof of the Caltech Jet Propulsion Laboratory building. The peak acceleration in the basement of the building was 20%g, and the peak acceleration on the roof of the building was 40%g.



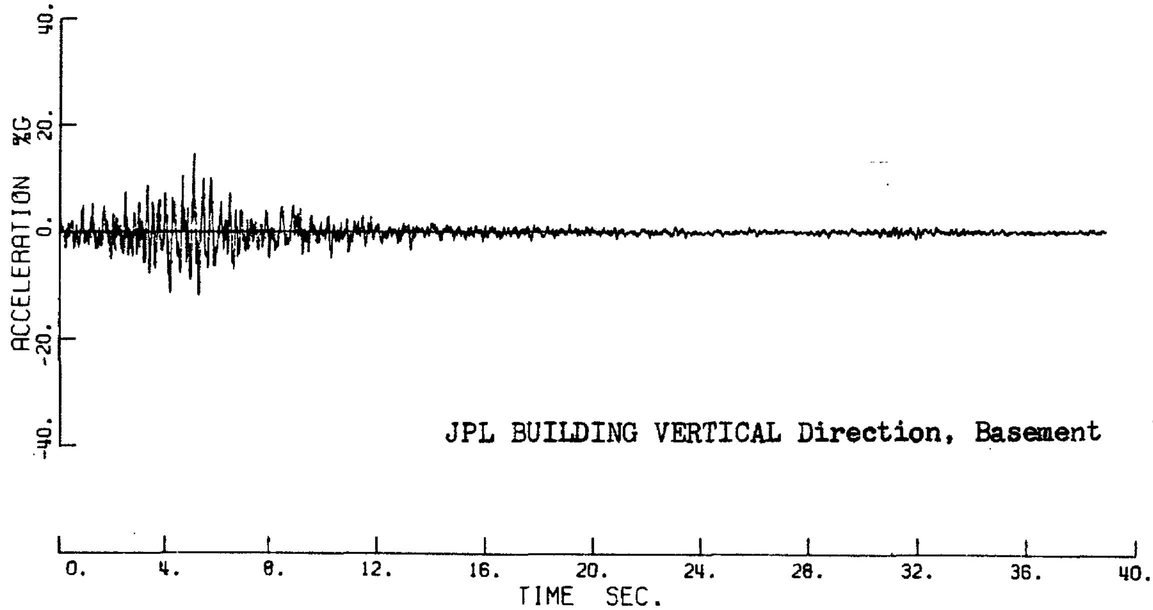
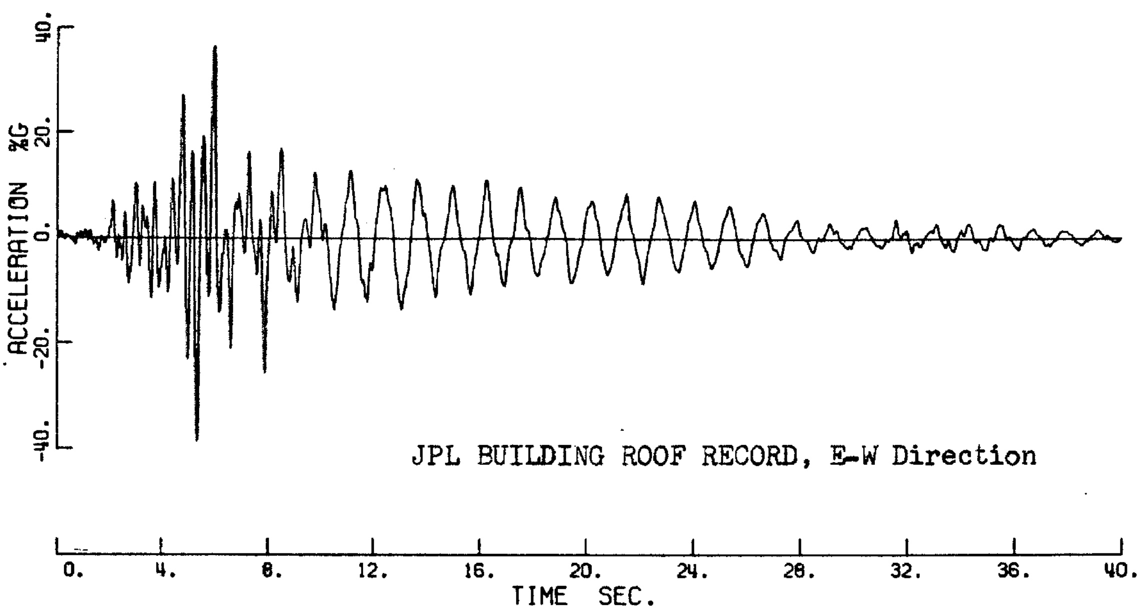


Figure 18. Vertical accelerations recorded in the basement and on the roof of the Jet Propulsion Laboratory building. The peak acceleration in the basement was 15%g, and the peak acceleration on the roof was 28%g.



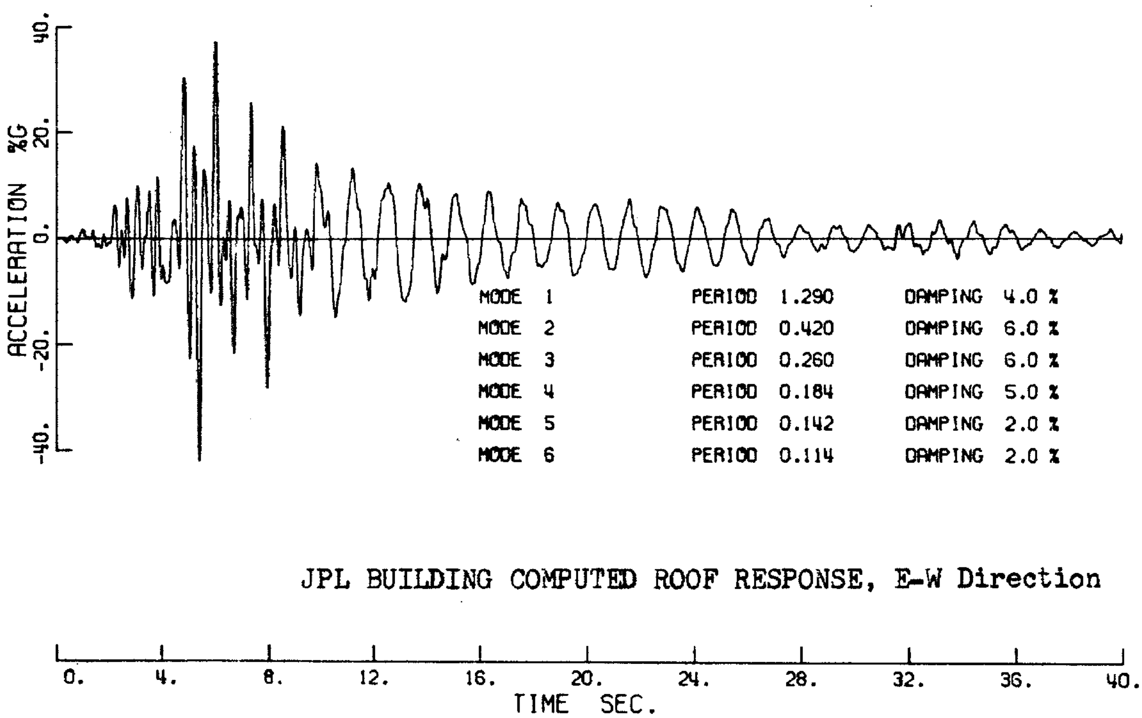
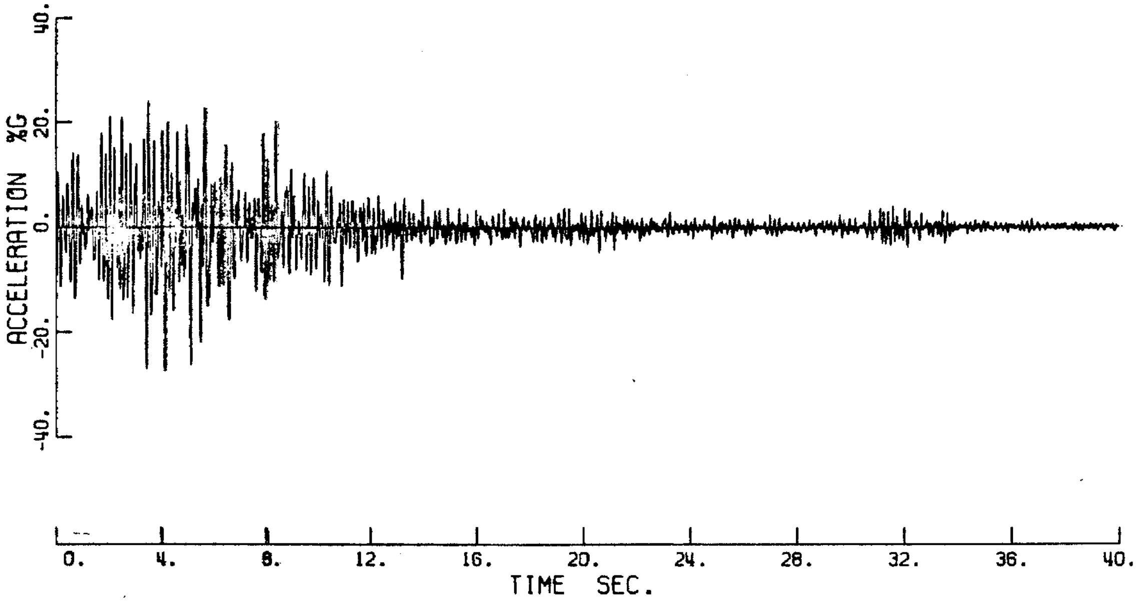
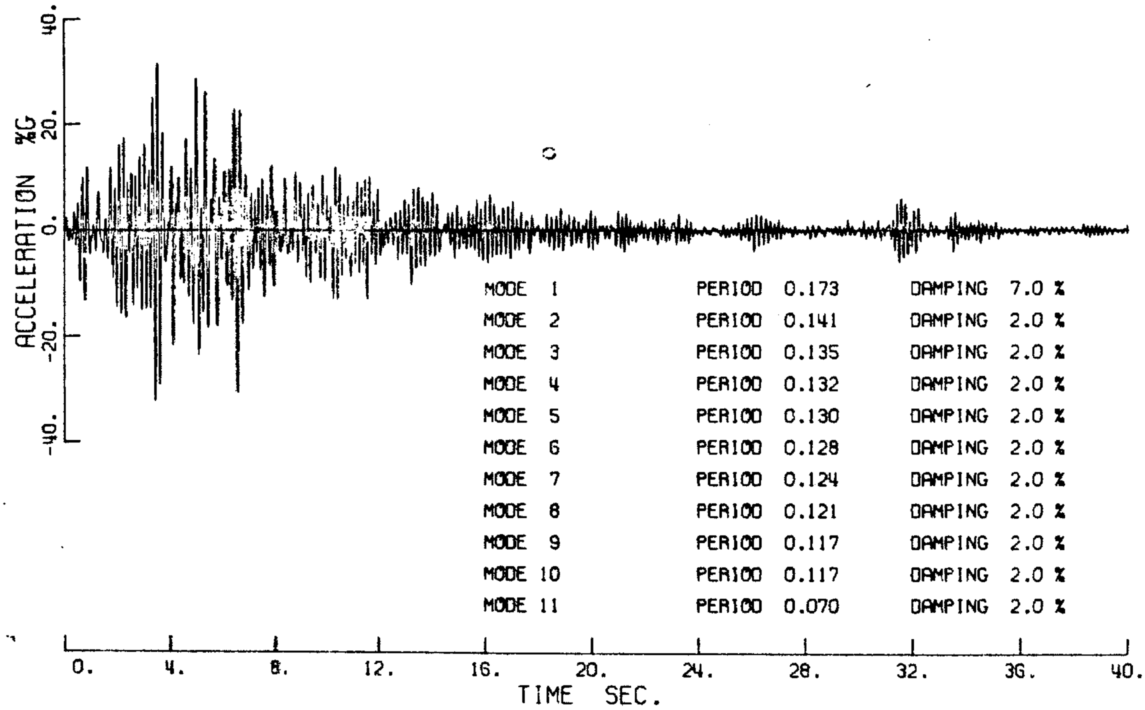


Figure 19. Comparison of the recorded roof accelerations and the computed roof accelerations. Taking the recorded base motion as input, by means of a digital computer the motion at the top of the building was computed.



JPL BUILDING ROOF RECORD, VERTICAL DIRECTION



JPL BUILDING COMPUTED ROOF RESPONSE, VERTICAL DIRECTION

Figure 20. Comparison of vertical motion recorded on the roof and the vertical motion computed for the roof.

rise buildings is given on page 53, Item 4(d)1.

11. Summary and Conclusions.

Research on the occurrence of earthquakes, on the nature of the ground motion generated by them, and on the motions of buildings during earthquakes provided information sufficient to formulate realistic earthquake design methods for high-rise buildings. Such advance methods of design were used for high-rise buildings in Los Angeles. The ground motions and building motions recorded during the 9 February 1971 San Fernando, California earthquake verified that these methods of design were, indeed, correct. As a consequence, the earthquake design requirements of the Los Angeles Building Code were changed so as to require that high-rise buildings be designed on the basis of a dynamic analysis in the future. This is a good example of how research can have a practical payoff.

APPENDIX I

EARTHQUAKE-RESISTANT DESIGN REQUIREMENTS IN UNIFORM BUILDING CODE - 1935

Sec. 2311 (a) LATERAL BRACING.

Every building or structure and every portion thereof, except Type V buildings of Group I occupancy which are less than twenty-five (25) feet in height, and minor accessory buildings, shall be designed and constructed to resist stresses produced by lateral forces as provided in this Section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the foundation, such force shall be proportioned to the total dead plus one-half (½) the vertical design live load, except for warehouses, in which case such force shall be proportioned to the total dead plus the total vertical live load. The force shall be assumed to come from any horizontal direction.

All bracing systems both horizontal and vertical shall transmit all forces to the resisting members and shall be of sufficient extent and detail to resist the horizontal forces provided for in this section and shall be located symmetrically about the center of mass of the building or the building shall be designed for the resulting rotational forces about the vertical axis.

Junctures between distinct parts of buildings, such as wings which extend more than twenty (20) feet from the main portion of the building, shall be designed at the juncture with other parts of the building for rotational forces, or the juncture may be made by means of sliding fragile joint having a minimum width of not less than eight (8) inches. The details of such joints shall be made satisfactory to the Building Inspector.

Horizontal Force Formula:

In determining the horizontal force to be resisted, the following formula shall be used:

$$F=CW$$

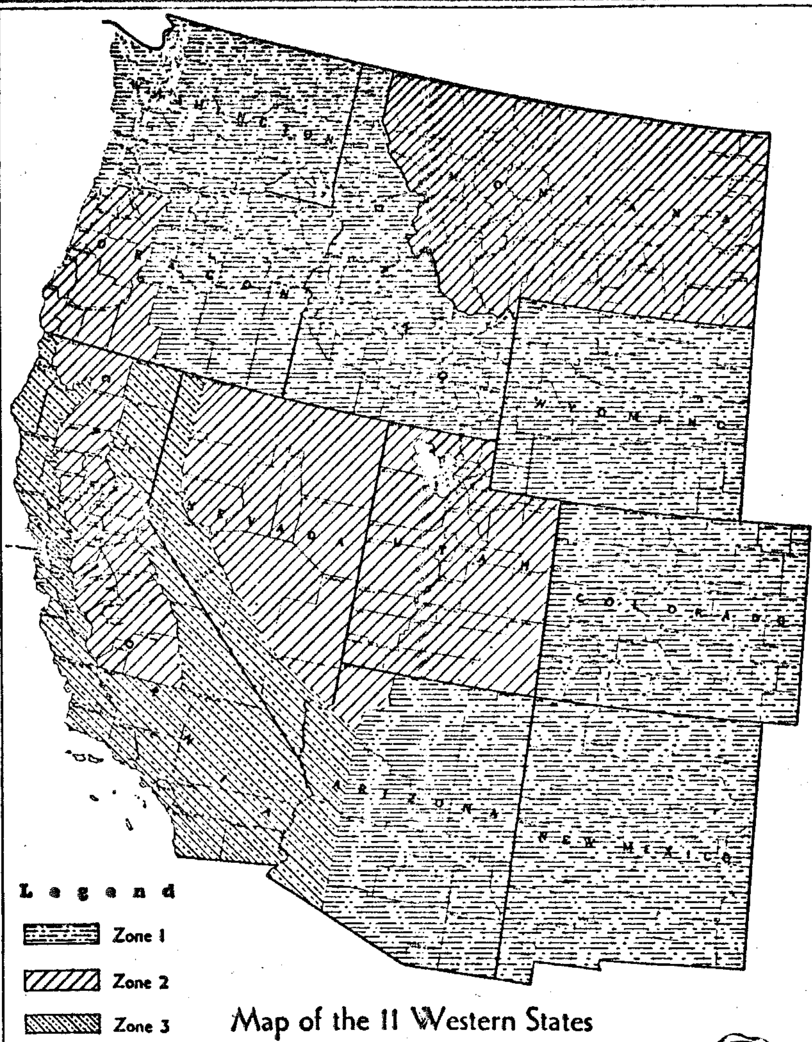
where "F" equals the horizontal force in pounds.
"W" equals the total dead load plus one-half (½) the total vertical designed live load,
at and above the point or elevation under consideration, except for warehouses, in which case "W" shall equal the total dead load plus the total vertical designed live load at and above the point or elevation under consideration. Machinery or other fixed concentrated loads shall be considered as part of the dead load.

"C" equals a numerical constant as shown in the following table:

Part or Portion	Value of "C"*	Direction of Force
The building as a whole**	.02 on soil, over 2000 lbs. .04 on soil, up to 2000 lbs.	Any horizontal direction
Bearing walls, curtain walls, enclosure walls, fire division walls, panel walls	.05	Normal to surface of wall
Cantilever parapet and other cantilever walls, except retaining walls.	.25	Normal to surface of wall
Exterior and interior ornamentations and appendages.	.25	Any direction horizontally
Towers, tanks, towers and tanks plus contents, chimneys, smoke stacks, and penthouses.	.05	Any direction horizontally

*See map on page 245 for zones. The values given "C" are minimum and should be adopted in locations not subjected to frequent seismic disturbances as shown in Zone 1. For locations in Zone 2, "C" should be doubled. For locations in Zone 3, "C" should be multiplied by four.

**Where a 20-lb. per square foot wind load would produce higher stresses, this load should be used in lieu of the factor shown.



Map of the 11 Western States

showing

Zones of Approximately Equal Seismic Probability

Foundation ties:

In the design of buildings of Types I, II and III, where the foundations rest on piles or on soil having a safe bearing value of less than two thousand (2,000) pounds per square foot, the foundations shall be completely inter-connected in two (2) directions approximately at right angles to each other. Each such inter-connecting member shall be capable of transmitting by both tension and compression at least ten (10) per cent of the total vertical load carried by the heavier only of the footings or foundations connected. The minimum gross size of each such member if of reinforced concrete shall be twelve inches by twelve inches (12"x12"). and shall be reinforced with not less than the minimum reinforcement specified in Section 2621. If the inter-connecting members are of structural steel, they shall be designed as provided in Section 2702, and encased in concrete. A reinforced concrete slab may be used in lieu of inter-connecting tie members, providing the slab thickness is not less than one forty-eighth (1/48) of the clear distance between the connected foundations; also providing the thickness is not less than six (6) inches.

The inter-connecting slabs shall be reinforced with not less than eleven-hundredths (.11) square inch of steel per foot of slab in a longitudinal direction and the same amount of steel in a transverse direction. The bottom of such slab shall not be more than twelve (12) inches above the tops of at least eighty (80) per cent of the piers or foundations. The footings and foundations shall be tied to the slab in such a manner as to be restrained in all horizontal directions.

Plans and Design Data:

With each set of plans filed, a brief statement of the following items shall be included:

(a) A summation of the dead and live load of the building, floor by floor, which was used in figuring the shears for which the building is designed.

(b) A brief description of the bracing system used, the manner in which the designer expects such system to act, and a clear statement of any assumptions used. Assumption as to location of all points of counter-flexure in members must be stated.

(c) Sample calculation of a typical bent or equivalent.

Stresses

Stresses in materials shall not exceed by more than thirty-three and one-third (33⅓) per cent the allowable working stresses permitted in this Code, except that rivets may be stresses the same in tension as is allowed in shear. Tension and/or shear in brick work shall not exceed twenty (20) pounds per square inch where cement mortar is used or fifteen (15) pounds per square inch where lime-cement mortar is used. The allowable shear in reinforced concrete walls, six (6) inches or more in thickness, shall not exceed five one-hundredths (.05) of the ultimate compressive strength of the concrete.

APPENDIX II

EARTHQUAKE-RESISTANT DESIGN REGULATIONS IN LOS ANGELES BUILDING CODE - 1970

SEC. 91.2305 — HORIZONTAL FORCES

(a) **General.** These lateral force requirements are intended to provide minimum standards as design criteria toward making buildings and other structures earthquake resistant. The provisions of this Section apply to the structure as a unit and also to all parts thereof, including the structural frame or walls, floor and roof systems, and other structural features.

Where the provisions of this Section are not applicable to special cases, the Department of Building and Safety may make interpretations necessary to fulfill the intent of this ordinance.

Every portion of every structure shall be designed to resist the lateral forces of wind or earthquake, whichever is greater.

EXCEPTION: The requirements of this Section shall not apply to any one-story, Type V building accessory to a dwelling.

Stresses shall be calculated as the effect of a force applied horizontally at each floor and each roof level above the foundation. The force shall be assumed to come from any horizontal direction.

(b) **Definitions.** The following definitions apply only to the provisions of this section:

➤ **Base:** is that level of the building or structure where the lateral load is effectively transferred to the ground. ◄

TABLE NO. 23-B
HORIZONTAL FORCE FACTOR "C_p" FOR PARTS OR PORTIONS OF BUILDINGS OR OTHER STRUCTURES

Part or Portion of Buildings	Direction of Force	Value of C_p
Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions over ten feet in height, masonry fences over six feet in height	Normal to Flat Surface	0.20
Cantilever parapet and other cantilever walls, except retaining walls	Normal to Flat Surface	1.00
Exterior and interior ornamentations and appendages	Any Direction	1.00
When connected to or a part of a building: towers, tanks, towers and tanks plus contents, chimneys, smokestacks, and penthouses	Any Direction	0.20(1)
Tanks plus effective contents resting on the ground	Any Direction	0.10
Floors and roofs acting as diaphragms	Any Direction	(2)
Prefabricated structural elements, other than walls, with force applied at center of gravity of assembly(3).	Any Horizontal Direction	0.30
➤Connections for exterior panels or elements complying with Section 91.2305 (k) 7	Any Direction	2.00 ◄

(1) When H/D of any building is equal to or greater than 5 to 1 increase value by 50%.
(2) Floors and roofs acting as diaphragms shall be designed for a minimum value of C_p of 10% applied to loads tributary from that story unless a greater value of C_p is required by the basic seismic formula $V = KCW$.
(3) The W_p shall be equal to the total dead load plus 25% of the floor live load in storage and warehouse occupancies.

TABLE NO. 23-C
HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES(1)

Type or Arrangement of Resisting Elements	Value of K
All building framing systems except as hereinafter classified.	1.00
Buildings with a box system as defined in Section 2305(b).	1.33
➤Buildings with a dual bracing system consisting of a ductile moment-resisting space frame and shear walls designed in accordance with the following criteria: 1. The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames. 2. The shear walls acting independently of the ductile moment-resisting space frame shall resist the total required lateral force. 3. The ductile moment-resisting space frame shall have the capacity to resist not less than 25% of the required lateral force.➤	0.80
➤ Buildings with a ductile moment-resisting space frame designed in accordance with the following criteria: 1. The ductile moment-resisting space frame shall have the capacity to resist the total required lateral force. 2. If major rigid elements are included in addition to the ductile moment-resisting space frame, the total required lateral force shall be distributed to all resisting elements in accordance with their relative rigidities considering the interaction of the frames and rigid elements.➤	0.67
➤Elevated tanks plus contents supported on four or more cross-braced columns and not supported by a building.	3.00(2) ➤
Structures other than buildings and other than those listed in Table 23-B.	1.50

(1) Where wind load as set forth in Subsection 2305(*l*) would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.
(2) Sec. 91.2305(d)4

Space Frame: is a three dimensional structural system composed of interconnected members, other than ➤ bearing walls, laterally supported so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.➤

Space Frame — Vertical Load-Carrying: A space frame designed to carry all vertical loads.

Space Frame—Moment-Resisting: ➤ is a vertical load-carrying space frame in which the members and joints are capable of resisting design lateral forces by bending moments.➤

➤**Lateral Force Resisting System:** is that part of the structural system to which the lateral forces prescribed in Subdivision 1 of Subsection (d) of this Section are assigned.➤

➤**Space Frame—Ductile Moment-Resisting:** is a Space Frame—Moment-Resisting complying with the requirements for a ductile moment-resisting space frame as given in Section 91.2305(j).➤

Box System is a structural system without a complete vertical load-carrying space frame. In this system the required lateral forces are resisted by shear walls as hereinafter defined.

Shear Wall is a wall designed to resist lateral forces parallel to the wall. Braced frames subjected primarily to axial stresses shall be considered as shear walls for the purpose of this definition.

(c) **Symbols and Notations.** The following symbols and notations apply only to the provisions of this Section.

C	= Numerical coefficient for base shear as defined in Section 2305(d)1.
C_p	= Numerical coefficient as defined in Section 2305 (d)2 and as set forth in Table No. 23-B.
D	= The dimension of the building in feet in a direction parallel to the applied force.
D_s	= Plan dimension of the vertical lateral force resisting system in feet.
F_a	= Allowable axial stress.
f_a	= Computed axial stress.
F_b	= Allowable bending stress.
f_b	= Computed bending stress.
F_i, F_n	= Lateral force applied to level i or n respectively.
F_p	= Lateral forces on the part of the structure and in the direction under consideration.
F_t	= That portion of V considered concentrated at the top of the structure, at the level n. The remaining portion of the total base shear V shall be distributed over the height of the structure, including level n.
F_x	= Lateral force applied to a level designated as x.
h_i, h_n	= The height in feet above the base to level i or n respectively.
h_x	= Height in feet above the base to the level designated as x.
J	= Numerical coefficient for base moment as defined in Section 2305(h).
J_x	= Numerical coefficient for overturning moment at level x.
K	= Numerical coefficient as set forth in Table No. 23-C.
Level i	= Level of the structure referred to by the subscript i.
Level n	= That level which is uppermost in the main portion of the structure.
Level x	= That level which is under design consideration.
M	= Overturning moment at the base of the building or structure.
M_x	= The overturning moment at level x.
N	= Total number of stories above exterior grade to level n.
T	= Fundamental period of vibration of the building or structure in seconds in the direction under consideration.
V	= Total lateral load or shear at the base $= (F_t + \sum_{i=1}^{n} F_i)$ where i = 1 designates first level above the base.
W	= Total dead load $= (\sum_{i=1}^{n} w_i)$ *EXCEPTION: W shall be equal to the total dead load plus 25% of the floor live load in storage and warehouse occupancies.*
W_p	= The weight of a part or portion of a structure.
w_i, w_x	= That portion of W which is located at or is assigned to level i or x respectively.

(d) **Minimum Earthquake Forces for Structures.** 1. **Total Lateral Force and Distribution of Lateral Force.** Every structure shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the structure in accordance with the following formula:

$$V = KCW$$

The value of K shall be not less than that exhibited in Table 23-C. The value of C need not exceed 0.10 and shall be determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}}$$

EXCEPTION: C shall be 0.10 for all one and two story buildings.

T is the fundamental period of vibration of the structure in seconds in the direction under consideration. Properly substantiated technical data for establishing the period T for the contemplated structure may be submitted.

In the absence of such data, the value of T for buildings shall be determined by the following formula:

$$T = \frac{0.05\, h_n}{\sqrt{D}}$$

EXCEPTION: In all buildings in which the lateral force resisting system consists of a moment-resisting space frame which resists 100% of the required lateral forces and which frame is not enclosed by, or adjoined by, more rigid elements which would tend to prevent the frame from resisting lateral forces, the period T shall be computed as follows:

$$T = 0.10\,N$$

The total lateral force V shall be distributed over the height of the structure in the following manner:

A portion F_t of the total lateral force V shall be concentrated at the top of the structure in accordance with the following:

$$F_t = 0.004 \left(\frac{h_n}{D_s} \right)^2 V$$

F_t need not exceed 0:15 V and may be considered as zero for values of $\left(\frac{h_n}{D_s} \right)$ of 3 or less, and

The remainder of the lateral force $(V - F_t)$ shall be distributed over the height of the structure (including the top level) in accordance with the following:

$$F_x = \frac{(V - F_t) w_x h_x}{\left(\sum_{i=1}^{n} w_i h_i \right)}$$

EXCEPTION: One and two-story buildings shall have uniform distribution.

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

2. **Lateral force on parts or portions of building or other structures.** Parts or portions of buildings or structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = C_p W_p$$

The values of "C_p" are in Table No. 23-B. The distribution of these forces shall be according to the gravity loads pertaining thereto.

3. **Pile foundations.** Individual pile or caisson footings of every building or structure shall be stayed in all directions at grade level by members capable of transmitting in tension and compression a force equal to 10% of the larger pile cap load. These members may be omitted where it can be demonstrated that an equivalent restraint can be provided by other means.

SEE RULE OF GENERAL APPLICATION #662 IN APPENDIX SECTION

4. **Elevated Tanks.**

A. Designs for elevated tanks on four or more cross-braced columns and not supported by a building shall conform to the following:

The period "T" shall be substantiated by technical data.

The value of "KC" as used in V = KCW in this Subsection shall not be less than 0.12 but need not exceed 0.25.

The factor "J" in the overturning analysis shall be 1.00.

Resistance to horizontal torsion shall be provided and the torsional eccentricity shall be not less than 5% as provided in this Section for buildings.

B. Designs for elevated tanks having arrangements of columns other than in Part A shall use a value of "KC" equal to not less than 0.20 and other provisions of Part A shall apply.

➔(e) **Distribution of Horizontal Shear.** Total shear in any horizontal plane shall be distributed to the various elements of the lateral force resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements that are assumed not to be part of the lateral force-resisting system may be incorporated into buildings provided that their effect on the action of the system is considered and provided for in the design. ←

(f) **Drift.** Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice.

(g) **Horizontal Torsional Moments.** Provisions shall be made for the increase in shear resulting from horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. In addition, where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than 5% of the maximum building dimension at that level.

➔ (h) **Overturning.** Every building or structure shall be designed to resist the overturning effects caused by the wind forces as set forth in Subsection (*l*) of this Section, or the earthquake forces specified in this Section, whichever governs.

***EXCEPTION:** The axial loads from earthquake force on vertical elements and footings in every building or structure may be modified in accordance with the following provisions:*

(1) The overturning moment "M" at the base of the building or structure shall be determined in accordance with the following formula:

$$M = J(F_t h_n + \sum_{i=1}^{n} F_i h_i)$$

Where $J = \dfrac{0.5}{\sqrt[3]{T^2}}$

The value of J need not be more than 1.00 *(h) The overturning moment "M_x" at any level designated as "x" shall be determined in accordance with the following formula:*

$$M_x = J_x \left[F_t (h_n - h_x) + \sum_{i=x}^{n} F_i (h_i - h_x)\right]$$

Where $J_x = J + (1 - J)\left(\frac{h_x}{h_n}\right)^3$

Seventy-five percent of the dead load may be used to reduce tensile stresses caused by seismic overturning moments.

At any level, the incremental changes of the design overturning moments, in the story under consideration, shall be distributed to the various resisting elements in the same proportion as the distribution of the shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, the overturning moment carried by the lowest story of that element shall be carried down as loads to the foundation.<

(i) Set-Backs. Buildings having set-backs wherein the plan dimension of the tower in each direction is at least 75% of the corresponding plan dimension of the lower part may be considered as a uniform building without set-backs for the purpose of determining seismic forces.

For other conditions of set-backs the tower shall be designed as a separate building using the larger of the seismic coefficients at the base of the tower determined by considering the tower as either a separate building for its own height or as part of the overall structure. The resulting total shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height.

>**(j) Structural Systems. 1. Design Requirements.** Buildings designed with a horizontal force factor "K" of 0.67 or 0.80 shall have a ductile moment-resisting space frame. Buildings more than 160 feet in height shall have a ductile moment-resisting space frame capable of resisting not less than 25% of the required seismic force for the structure as a whole.

Moment-resisting space frames and ductile moment-resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

2. Construction. The necessary ductility for a ductile moment-resisting space frame shall be provided by a frame of structural steel conforming to ASTM A-7, A-36 or A-441 with moment-resisting connections, or by a reinforced concrete frame complying with Section 91.2670 of this Code.

Shear walls in buildings exceeding one hundred and sixty feet in height shall be composed of axially loaded bracing members of ASTM A-7, A-36 or A-441 structural steel; or reinforced concrete bracing members or walls conforming with the requirements of Section 91.2680 of this Code. <

(k) Design Requirements. 1. Combined axial and bending stresses in columns. Except > for reinforced concrete columns designed in accordance with Division 26 of this Code and except for structural steel columns designed in accordance with Division 27 of this Code, all structural columns shall conform to this subdivision. The maximum allowable extreme fiber stress in columns at intersection of columns with floor beams or girders for combined axial and bending stress shall be the allowable bending stresses for the material used. Within the center one-half of the unsupported length of the column, the combined axial and bending stresses shall be such that:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \text{ is equal to or less than 1.}$$

When stresses are due to a combination of vertical and lateral loads, the allowable unit stresses may be increased as specified in Subsection 91.2301(g). <

2. **Building Separations.** All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance of at least one inch, plus ½ inch for each 10 feet of height above 20 feet.

3. **Minor Alterations.** Minor alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this section of the Code.

>4. **Reinforced Masonry or Concrete.** All elements within a structure which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete under the provisions of Divisions 24 and 26. Principal reinforcement in masonry shall be spaced four feet maximum on center except that a maximum spacing of two feet on center shall be used in buildings utilizing a ductile moment-resisting space frame. <

5. **Combined Vertical and Horizontal Forces.** In computing the effect of seismic force in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load, except roof live load, shall be considered.

>6. **Minor Rigid Elements.** Minor rigid elements within or attached to a structure may be assumed to be expendable and not part of the lateral force resisting system.

7. **Exterior Elements.** Precast, non-bearing, non-shear wall panels or other elements which are attached to, or enclose the exterior, shall accommodate movements of the structure resulting from lateral forces or temperature changes. The concrete panels or other elements shall be supported by means of poured-in-place concrete or by mechanical fasteners in accordance with the following provisions:

A. Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind or seismic forces; or ¼-inch whichever is greater.

B. Connections shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds. Inserts in concrete shall be attached to, or hooked around reinforcing steel, or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

C. Connections to permit movement in the plane of the panel for story drift may be properly designed sliding connections using slotted or oversize holes or may be connections which permit movement by bending of steel. <

(l) **Wind Pressures.** The amount of wind pressure shall be assumed to be not less than the values exhibited in Table No. 23-D.

APPENDIX III

SEISMIC DESIGN CODE CHANGES
CITY OF LOS ANGELES - MAY 1973

These changes include revisions recommended by the Building Department as a result of the public hearing of July 31, 1972. (These changes will be incorporated in the next printing of the building code).

ITEM 1

Section 91.2305, subsection (j) of the Los Angeles Municipal Code is to be amended to read as follows:

(j) Structural Systems. ~~1. Design Requirements. Building designed with a horizontal force factor "K" of 0.67 or 0.80 shall have a ductile moment resisting space frame. Buildings more than 160 feet in height shall have a ductile moment resisting space frame capable of resisting not less than 25% of the required seismic force for the structure as a whole.~~

~~Moment-resisting space frames and ductile moment resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.~~

1. Special Requirements.

A. All buildings designed with a horizontal force factor "K" = 0.67 or 0.80 shall have space frames-ductile moment resisting.

B. Buildings more than 160 feet in height shall have space frames - ductile moment resisting capable of resisting not less than 25% of the required seismic forces for the structure as a whole.

C. All concrete space frames required by design to be part of the lateral force resisting system and all concrete frames located in the perimeter line of vertical support shall be space frames - ductile moment resisting.

EXCEPTION: Frames in the perimeter line of vertical support of buildings designed with shear walls taking 100% of the design lateral forces need only be checked for conformance with the following sub-item (D).

D. All framing elements not required by design to be part of the lateral force resisting system shall be investigated and shown to be adequate for vertical load and induced moment due to four times the distortions resulting from the code required lateral forces. The rigidity of other elements shall be considered in accordance with Section 91.2305(e).

E. Moment-resisting space frames and ductile moment-resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frames from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

2. Construction. A ductile moment-resisting space frame shall consist of a moment-resisting frame of structural steel conforming to Section 91.2722 of this Code, or by a reinforced concrete frame complying with Section 91.2670 of this Code.

Shear walls in buildings where "K" equals 0.80 shall be composed of axially loaded bracing members of ASTM A36, A440, A441, A572 (except Grades 60 and 65) or A588 Grades A, B or C structurall steel, or reinforced concrete as provided in the following paragraph.

Reinforced concrete shear walls and reinforced concrete braced frames for all buildings shall conform to the requirements of Section 91.2680 of this Code.

In buildings where "K" equals 0.67 or 0.80, all structural elements below the base required to transmit lateral forces to the foundation shall be composed of structural steel complying with Section 91.2722 or of reinforced concrete complying with Sections 91.2670 and 91.2680.

ITEM 2

Section 91.2670 of the Los Angeles Municipal Code is to be repealed and a new Section 91.2670 adopted to read as follows: (for clarity the present Section to be repealed is not shown and the new Section is not underlined.)

Section 91.2670. Concrete Ductile Moment-Resisting Space Frames.

(a) General. 1. Design and construction of cast-in-place, monolithic reinforced concrete framing members and their connections in ductile moment resisting space frames shall conform to the requirements of A.C.I. Building Code, A.C.I. 318, and all the requirements of this Section.

EXCEPTION: Precast concrete frame members may be used, if the resulting construction complies with all the provisions of this section.

2. All lateral load-resisting frame members shall be designed by the ultimate strength method except that the working stress design method may be used provided that it is shown that the factor of safety is equivalent to that achieved with the ultimate strength design method.

3. A.C.I. 318, for earthquake loading shall be modified to:

$$U = 1.40\ (D+L+E) \qquad (26\text{-}33)$$

$$U = .90\ D+1.40E \qquad (26\text{-}34)$$

4. Members of space frames which are designed to resist seismic forces shall be designed, in accordance with the provisions of this section, so that shear failures will not occur if the frame is subjected to lateral displacements in excess of yield displacements.

(b) Definitions. 1. Confined Concrete. Concrete which is confined by closely spaced special transverse reinforcement which is provided to restrain the concrete in directions perpendicular to the applied stresses.

2. Special Transverse Reinforcement. Spirals, stirrup ties, or hoops and supplementary crossties provided to restrain the concrete to make it qualify as confined concrete.

3. Stirrup-ties or Hoops. Continuous reinforcing steel of of not less than a No. 3 bar bent to form a closed hoop which encloses the longitudinal reinforcing and the ends of which have a standard 135 degree bend with a 10 bar diameter extension or equivalent.

(c) Physical Requirements for Concrete and Reinforcing Steel.

1. Concrete. The minimum specified 28-day strength of the concrete, f_c' shall be 3000 pounds per square inch.

The maximum specified strength for lightweight concrete shall be limited to 4000 psi.

2. Reinforcement. All longitudinal reinforcing steel in columns and beams shall comply with ASTM A-615, grade 40 or 60. The actual yield stress, based on mill tests, shall not exceed the minimum specified yield stress, f_y, by more than 18,000 psi. Retests shall not exceed this value by more than an additional 3000 psi. In addition the ultimate tensile stress shall be not less than 1.33 times the actual yield stress, based on mill tests. Grades other than these specified for design shall not be used.

Where reinforcing steel is to be welded, a chemical analysis of the steel shall be provided. The welding procedure shall be set forth in the American Welding Society's publication, AWS D12.1 "Recommended Practice for Welding Reinforcing Steel, Metal Inserts and Connections in Reinforced Concrete Construction."

(d) Flexural Members. 1. General. Flexural members shall not have a width-depth ratio of less than 0.3, nor shall the width be less than 10 inches nor more than the supporting column width plus a distance on each side of the column of three-fourths the depth of the flexural member . Flexural members framing into columns shall be subject to a rational joint analysis.

2. Reinforcement. All flexural members shall have a minimum reinforcement ratio, for top and for bottom reinforcement, of $\frac{200}{f_y}$ throughout their length. The reinforcement ratio, p, shall not exceed 0.025.

The positive moment capacity at the face of columns shall be not less than 50 per cent of the negative moment capacity provided. A minimum of one-fourth of the larger amount of the negative reinforcement required at either end shall continue throughout the length of the beam. At least two bars shall be provided both top and bottom.

3. Splices. Tensile steel shall not be spliced by lapping in a region of tension or reversing stress unless the region is confined by stirrup-ties. Splices shall not be located within the column or within a distance of twice the member depth from the face of the column. At least two stirrup-ties shall be provided at all splices.

4. Anchorage. Flexural members terminating at a column, in any vertical plane, shall have top and bottom reinforcement extending, without horizontal offsets, to the far face of a confined concrete region (Section 91.2670 (e)4) terminating in a standard 90 degree hook. Length of required anchorage shall be computed beginning at the near face of the column. Length of anchorage in confined regions shall be determined by:

$$L = \frac{A_s f_y}{1.5 u_u \Sigma_o} \qquad (26\text{-}35)$$

including hook and vertical extension, but not less than 24 inches.

EXCEPTION: Where the column resists less than 25% of the story-bent shear, at least 50% of such top and bottom reinforcement shall be anchored within such column cores and the remainder shall be anchored in regions outside the column core confined as specified herein for columns.

5. Web Reinforcement. Vertical web reinforcement of not less than NO. 3 bars shall be provided in accordance with the requirements of A.C.I. 318, except that:

a. Maximum

$$V_u \geq \frac{M_u^A + M_u^B}{L} + 1.4V_{D+L} \qquad (26\text{-}36)$$

where M_u^A and M_u^B are ultimate moment capacities of opposite sense at each end of the member and V_{D+L} is the simple span shear. Ultimate moment capacities shall be computed without the ϕ factor reduction and assuming the maximum reinforcing yield strength based on 25% over specified yield. Ultimate shear capacities shall be computed with the ϕ factor reduction.

b. Stirrups shall be spaced at no more than d/2 throughout the length of the member.

c. Stirrup-ties, at a maximum spacing of not over d/4, 8 bar diameters, 24 stirrup-tie diameters, or 12 inches, whichever is least, shall be provided in the following locations:

1. At each end of all flexural members. The first stirrup-tie shall be located not more than 2 inches from the face of the column and the last, a distance of at least twice the member depth from the face of the columns.

2. Wherever ultimate moment capacities may be developed in the flexural members under inelastic lateral displacement of the frame.

3. Wherever required compression reinforcement occurs in the flexural members.

d. In regions where stirrup-ties are required, longitudinal bars shall have lateral support conforming to the provisions of ties for tied columns. Single or overlapping stirrup-ties and supplementary crossties may be used.

(e) Columns Subject to Direct Stress and Bending. 1. Dimensional limitations. The ratio of minimum to maximum column thickness shall not be less than 0.4 nor shall any dimension be less than 12 inches.

2. Vertical Reinforcement. The reinforcement ratio, p, in tied columns shall be not less than 0.01 nor greater than 0.06.

3. Splices. Lap splices shall be made within the center half of column height, and the splice length shall not be less than 30 bar diameters. Continuity may also be effected by welding or by approved mechanical devices provided not more than alternate bars are welded or mechanically spliced at any level and the vertical distance between these welds or splices of adjacent bars is not less than 24 inches.

4. Special Transverse Reinforcement. The cores of columns shall be confined by special transverse reinforcement as specified herein or as required to meet shear requirements.

a. The volumetric ratio of spiral reinforcement shall not be less than that required in ACI 318 nor

$$p''=0.12\frac{f'_c}{f''_{yh}}, \qquad (26\text{-}37)$$

whichever is greater.

b. The total cross-sectional area (A''_{sh}) of rectangular hoop reinforcement shall not be less than

$$A''_{sh}=0.30ah''\frac{f'_c}{f''_{yh}}\left(\frac{A_g}{A_c}-1\right), \qquad (26\text{-}38)$$

nor

$$A''_{sh}=0.12ah''\frac{f'_c}{f''_{yh}}, \qquad (26\text{-}39)$$

whichever is greater, where

a = center to center spacing of hoops in inches with a maximum of 4 inches.

A_c = area of column core.

A_g = gross area of column.

A''_{sh} = total cross-sectional area in square inches of hoop reinforcement including supplementary crossties having a spacing of (a) inches and crossing a section having a core dimension of h".

h" = core dimension of tied column inches.

f''_{yh} = yield strength of hoop or spiral reinforcement.

Single or overlapping hoops may be provided to meet this requirement.

Supplementary crossties of the same size and spacing as hoops using 135° minimum hooks engaging the periphery hoop and secured to a longitudinal bar may be used. Supplementary crossties or legs of overlapping hoops shall not be spaced more than 14" on center transversely.

EXCEPTION: Equation (26-38) need not be complied with if the column design is based on the column core only.

c. Special transverse reinforcement shall be provided in that portion of the column over a length equal to the maximum column dimension or one-sixth of the clear height of the column, but not less than 18 inches from either face of the joint.

d. At any section where the ultimate capacity of the column (P_u) is less than the sum of the shears (ΣV_u) computed by Equation (26-36) for all the beams framing into the column above the level under consideration, special transverse reinforcement shall be provided. For beams framing into opposite sides of the column, the moment components of Equation (26-36) may be assumed to be of opposite sign. For the purpose of this determination the factor of 1.4 in Equation (26-36) may be changed to 1.1. For determination of P_u, the moments resulting from Equation (26-36) may be assumed to result from deformation of the frame in any one principal axis.

e. Columns which support discontinuous members, such as shear walls, braced frames, or other rigid elements shall have special transverse reinforcement for the full height of the supporting columns.

5. Column Shear. The transverse reinforcement in columns subjected to bending and axial compression shall satisfy the following requirement:

$$A_v f_y \frac{d}{s} = V_u - V_c \qquad (26\text{-}40)$$

WHERE

V_u, the maximum ultimate shear, shall be computed by using the yield moments in the ends of either the beams or columns framing into the connection. Ultimate moment capacities shall be computed without ϕ or other reduction factors and under all possible vertical loading conditions and assuming the maximum reinforcing yield strength based on 25% over specified yield. Ultimate shear capacity shall be computed with the ϕ factor reduction and shall be based on the column core area for shear resistance.

$V_c = v_c bd$, where v_c shall be in accordance with A.C.I. 318, except that v_c shall be considered zero when $\frac{P}{A_g} < 0.12f'_c$.

s = spacing, $< \frac{1}{2}$ minimum column dimension.

d = effective depth of section.

A_v = total cross sectional area of special transverse reinforcement in tension within a distance(s), except that two-thirds of such area shall be used in the case of circular spirals.

(f) Beam-Column Connection. Special transverse reinforcement shall be provided through the beam column connection.

1. Analysis. The transverse reinforcement through the connection shall be proportioned according to the requirements of Section 91.2670(e)4. The transverse reinforcement thus selected shall be checked according to the provisions set forth in Section 91.2670(e)5 with the exception that the V_u acting on the connection shall be equal to the maximum shears in the connection computed by a rational analysis taking into account the column shear and the concentrated shears developed from the forces in the beam reinforcement at a stress assumed at f_y.

2. Special transverse column reinforcement of one-half the amount otherwise required by Subsection (f)1 shall be required within the connection, determined by the depth of the shallowest framing member, where such members frame into all four sides of a column and whose width is at least three-fourths the column width. When a corner of a tied column, unconfined by flexural members, exceeds 4 inches, the full special transverse reinforcement shall be provided through the connection and around bars outside of the connection.

3. Special transverse beam reinforcing shall be provided through the beam column connection to provide confinement for longitudinal reinforcement outside the column core where such confinement is not provided by another beam framing into the connection.

4. Design Limitations. At any beam-column connection where

$$\frac{P}{A_g} \geq 0.12f'_c$$

the total ultimate moment capacity of the column, at the design earthquake axial load, shall be greater than the total ultimate moment capacity of the beams, along their principal planes at that connection.

EXCEPTION: Where certain beam-column connections at any level do not comply with the above limitations, the remaining columns and connected flexural members shall comply and further shall be capable of resisting the entire shear at that level accounting for the altered relative rigidities and torsion resulting from the omission of elastic action of the non-conforming beam column connections.

ITEM 3

Section 91.2680, subsection (a), subdivision 3 of the Los Angeles Municipal Code is to be amended as follows:

3. Equations (15-2) and (15-3) A.C.I. 318 for earthquake loading shall be modified to:

~~U = 1.4(D+L+E)~~	~~(15-2A)~~
$U = 1.4(D+L)+1.4E$	(26-41)
~~U = 0.9D+1.25E~~	~~(15-3A)~~
$U = 0.9D+1.4E$	(26-42)

provide further that ~~twice the "U" value set forth above~~ 2.8E shall be used in both equations in calculating shear and diagonal tension in buildings without a 100% moment-resisting space frame.

ITEM 4

Revise Section 91.2305(d) as follows:

(d) Minimum Earthquake Forces for Structures.

1. Dynamic Analyses. Every structure shall have structural capacity sufficient to resist the effects of earthquakes as determined by a dynamic analysis. This analysis shall be based on the ground shaking prescribed for the site in a geology-seismology report. Every geology-seismology report shall be subject to review and approval by the Department. Reports not approved by the Department shall not be used for the design of any structure.

EXCEPTION: Structures 160 feet or less in height, essentially regular in shape and in stiffness over their height, may be designed for earthquake force specified in Subdivision 2 of this Subsection.

2. Static Analyses. A General. Any structure 160 feet or less in height, essentially regular in shape and stiffness over its height, may be designed under the provisions of this Subdivision.

All steel bracing members and their connections used to resist seismic forces as determined under the provisions of this Subdivision shall be designed for 1.5 times the forces so determined.

Essential Facilities structures as defined in Subdivision 3 of this Subsection, shall be designed for 1.5 times the earthquake forces specified in this Subdivision.

Structures which are not essentially regular in plan dimension or which have significant changes in stiffness between stories shall be designed under the provision of Subdivision 1 of this Subsection.

~~1. Total Lateral Forces and Distribution of Lateral Force. Every structure shall be designed and constructed to withstand minimum total lateral.~~

B. Design. The seismic forces shall be assumed to act nonconcurrently in the direction of each of the main axes of the structure in accordance with the following formula:

$$V = KCW$$

The value of K shall be not less than that exhibited in Table 23-C. The value of C need not exceed 0.10 and shall be determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}}$$

EXCEPTION: C shall be 0.10 for all one- and two-story buildings.

T is the fundamental period of vibration of the structure in seconds in the direction under consideration. Properly substantiated technical data for establishing the period T for the contemplated structure may be submitted.

In the absence of such data, the value of T for buildings shall be determined by the following formula:

$$T = \frac{0.05\ h_n}{\sqrt{D}}$$

EXCEPTION: In all buildings in which the lateral force resisting system consists of a moment-resisting space frame which resists 100% of the required lateral forces and which frame is not enclosed by, or adjoined by, more rigid elements which would tend to prevent the frame from resisting lateral forces, the period T shall be computed as follows:

$$T = 0.10\ N$$

The total lateral force V shall be distributed over the height of the structure in the following manner:

A portion F_t of the total lateral force V shall be concentrated at the top of the structure in accordance with the following:

$$F_t = 0.004\left(\frac{h_n}{D_s}\right)^2 V$$

F_t need not exceed 0.15 V and may be considered as zero for

$$\text{values of } \left(\frac{h_n}{D} \right) \text{ of 3 or less, and}$$

The remainder of the lateral force $(V - F_t)$ shall be distributed over the height of the structure (including the top level) in accordance with the following:

$$F_x = \frac{(V - F_t)\ w_x h_x}{\left(\sum_{i=1}^{n} w_i h_i \right)}$$

EXCEPTION: One- and two-story buildings shall have uniform distribution.

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

C. ~~2.~~ Lateral force on parts or portions of building or other structures. Parts or portions of buildings or structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = C_p\ W_p$$

The values of "C_p" are in Table No. 23-B. The distribution of these forces shall be according to the gravity load pertaining thereto.

~~3. Pile foundations. Individual pile or caisson footings every building or structure shall be stayed in all directions at grade level by members capable of transmitting in tension and compression a force equal to 10% of the larger pile cap load. These members may be omitted where it can be demonstrated that an equivalent restraint can be provided by other means.~~

D. ~~4.~~ Elevated Tanks.

(1) Designs for elevated tanks on four or more cross-braces column and not supported by a building shall conform to the following:

The period "T" shall be substantiated by technical data.

The value of "KC" as used in V = KCW in this Subsection shall not be less than 0.12 but need not exceed 0.25.

Resistance to horizontal torsion shall be provided and the torsional eccentricity shall be not less than 5% as provided in this Section for buildings.

(2) Designs for elevated tanks having arrangments of columns other than in Part (1)shall use a value of "KC" equal to not less than 0.20 and other provisions of Part(1) shall apply.

3. Essential Facilities. Essential Facilities are those structures or buildings which must be safe and usable for emergency purposes after an earthquake in order to preserve the peace, health and safety of the general public. Such facilities shall include the following:

A. Hospitals and other medical facilities having surgery or emergency treatment areas;

B. Fire and police stations;

C. Municipal government disaster operation and communication centers:

Building elements and necessary equipment of Essential Facilities shall be designed, detailed and constructed to withstand the maximum acceleration and deflections of the structure without disrupting the operations or services.

evise Section 91.2305(i) as follows:

(i) Set-Backs Buildings Not designed With Dynamic Analysis.

Buildings having set-backs wherein the plan dimension of the tower in each direction is at least 75% of the corresponding plan dimension of the lower part may be considered as a uniform building without set-backs for the purpose of determining seismic forces.

For other conditions of set-backs the tower shall be designed as a separate building using the larger of the seismic coefficients at the base of the tower determined by considering the tower as either a separate building for its own height or as part of the overall structure. The resulting total shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height.

Revise Section 91.2305(k) by adding a new Subsection 8.

8. ~~3.~~ Pile foundations. Individual pile or caisson footings of every building or structure shall be stayed in all directions at grade level by members capable of transmitting in tension and compression a force equal to 10% of the larger pile cap load. These members may be omitted where it can be demonstrated that an equivalent restraint can be provided by other means.

ITEM 5

Revise Table No. 24-H by adding a new note number (5) as follows:

TABLE NO. 24-H - MAXIMUM WORKING STRESSES IN POUNDS PER SQUARE INCH FOR REINFORCED SOLID AND HOLLOW UNIT MASONRY (1)

Type of Stress	Special Inspection Required	
	Yes	No
Compression-Axial, Walls	See Section 91.2418	One-half of the value permitted under Section 91.2418
Compression-Axial Columns	See Section 91.2420	One-half of the value permitted under Section 91.2420
Compression-Flexural	.33 f'm but not to exceed 900	.166 f'm but not to exceed 450
Shear: (5)		
No shear reinforcement (2)	.02 f'm but not to exceed 50	15
Reinforcement taking entire shear:		
Flexural members	.05 f'm but not to exceed 120	50
Shear Walls	.04 f'm but not to exceed 75	30
Modulus of Elasticity (3)	1000 f'm but not to exceed 3,000,000	500 f'm but not to exceed 1,500,000
Modulus of Rigidity (3)	400 f'm but not to exceed 1,200,000	200 f'm but not to exceed 600,000
Bearing on full area (4)	.25 f'm but not to exceed 900	.125 f'm but not to exceed 450
Bearing on 1/3 or less of area	.30 f'm but not to exceed 1200	.15 f'm but not to exceed 600
Bond -Plain bars	60	30
Bond-Deformed	140	100

NOTES: (See next page)

TABLE NO. 24-H (con't)

NOTES: (1) Stresses for hollow unit masonry are based on NET section.

(2) Web reinforcement shall be provided to carry the entire shear in excess of 20 pounds per square inch wherever there is required negative reinforcement and for a distance of one-sixteenth the clear span beyond the point of inflection.

(3) Where determinations involve rigidity consideration in combination with other materials or where deflections are involved, the moduli of elasticity and rigidity under columns entitled "yes" for special inspection shall be used.

(4) This increase shall be permitted only when the least distance between the edges of the loaded and unloaded areas is a minimum of one-fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonable concentric area greater than one-third, but less than the full area, shall be interpolated between the values given.

(5) Shear walls which resist seismic forces shall be designed to resist 1.5 times the forces as determined by Section 91.2305(d).

ITEM 6

Revise Section 91.2305(k)2 to read:

2. Building Separations. All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance of ~~at least~~ one inch, plus 1/2 inch for each 10 feet of height ~~above 22 feet~~ except where analysis shows additional separation is required.

All portions of structures shall have a minimum clearance from the adjacent property of not less than 1/2 inch for each 10 feet of height.

ITEM NO. 7

Revise Section 91.2680(c) as follows:

~~(c) Shear and Diagonal Tension Ultimate Strength Design.~~

~~1. The nominal ultimate shear stress in a concrete shear wall shall be computed by:~~

$$\cancel{v_u = \frac{V_u}{bd}}$$

~~The shear stress v_u shall not exceed~~

$$\cancel{v_u = \left[0.8 + 4.6 \left(\frac{H}{B}\right)\right] \phi \sqrt{f'_c}}$$

~~For purposes of this Subsection (c), H is the total height to which the shear wall extends in the structure, and B is the width of the wall in the direction of the shear force.~~

~~The value for v_u shall not exceed $10\phi \sqrt{f'_c}$ for H/B ratios greater than 3 and $5.4\phi \sqrt{f'_c}$ for H/B ratios less than one.~~

~~2. The total shear carried by a reinforced concrete wall shall be determined in accordance with the following:~~

$$\cancel{V_u = v_c bd + V'_u}$$

~~3. The shear stress carried by the concrete shall not exceed~~

$$\cancel{v_c = \left[3.7 - \frac{H}{(B)}\right] \phi \sqrt{f'_c}}$$

~~except that v_c shall not exceed $5.4\phi \sqrt{f'_c}$ for H/B ratios less than one and the minimum value need not be less than $3\phi \sqrt{f'_c}$ for H/B ratios greater than 2.7.~~

~~When structural lightweight concrete is used the limiting value of v_c as set forth above shall be multiplied by the factor of $0.15F_{sp}$.~~

~~The-area-of-shear-reinforcement-required-to-resist-the portion-of-the-shear-V_u^+ shall-be-computed-by:~~

$$A_v = \frac{V_u^+ s}{\phi f_y d\left(\frac{H}{D} - 1\right)}$$

~~but-in-no-case-will-the-reinforcement-be-less-than-required in-Section-91.2650-or-by:~~

$$A_v = \frac{V_u^+ s}{\phi f_y d}$$

(c) Shear and Diagonal Tension-Ultimate Strength Design.

1. The nominal ultimate shear stress resulting from forces acting parallel to shear walls shall be computed by:

$$v_u = \frac{V_u}{A_c}$$

Where: V_u = Ultimate shear computed according to Subsection 91.2305(e) and including the effect of gravity loads.

A_c = Area of concrete sections resisting V_u, sq. in.

2. The ultimate shear stress, v_u, thus computed shall not exceed that given by:

$$v_u = 2\phi \sqrt{f'_c} + \phi p f_y$$

Where "p" is the ratio of the area of reinforcement to the area of concrete section resisting the shear V_u. At least an equal percentage of reinforcement, p, shall be provided perpendicular to that required by this Subsection.

The average horizontal shear, v_u, for all wall piers sharing a common lateral force component shall not exceed $8\phi \sqrt{f'_c}$ and the v_u in any of the individual wall piers shall be not more than $10\phi \sqrt{f'_c}$. The value of the vertical shear v_u shall not exceed $10\phi \sqrt{f'_c}$ for horizontal wallelements.

3. The minimum reinforcing ratio "p" for all walls designed to resist Code seismic forces acting parallel to

the wall shall be .0025 each way. The maximum spacing of reinforcement each way shall not exceed d/3 or 18", whichever is smaller, where "d" is the dimension of the wall element parallel to the shear force. That portion of the wall reinforcement required to resist design shears shall be uniformly distributed.

4. Wall reinforcement required to resist wall shear shall be terminated with not less than a 90° bend plus a 6 bar diameter extension beyond the boundary reinforcing at vertical and horizontal end faces of wall sections. Wall reinforcement terminating in boundary columns or beams shall be fully anchored into the boundary elements.

ITEM 8

Revise Section 91.2301(g) as follows:

(g) Increase in Stresses.

All allowable stresses and soil-bearing values specified in this Code for working stress design may be increased 1/3 when considering wind or earthquake forces from a Static Analysis as per Section 91.2305(g)2, either alone or when combined with vertical loads. No increase will be allowed for vertical loads acting alone or for the stresses in connections resulting from earthquake forces.

Load factors for ultimate strength design of concrete and plastic design of steel shall be as indicated in the appropriate sections on material.

Wind and earthquake loads need not be assumed to act simultaneously.

APPENDIX IV

1973 Revisions to Structural Engineers Association of California Blue Book and Pertinent Changes to 1970 Uniform Building Code. Given in May 1973 Newsletter of SEAOC.

The SEAOC Seismology Committee is in the process of publishing 1) a Commentary for the modifications contained in Appendix F, 2) Appendix G containing 1973 revisions and Commentary and 3) an updated Blue Book which incorporates all Appendix F and G changes in the main body of the code. The 1973 edition of the UBC will also be off the press within a month or so.

The Seismology Committee feels the 1973 changes to the Blue Book and certain changes to the UBC are of sufficient importance that SEAOC members should be informed of them without waiting for publication of the revised Blue Book and the 1973 UBC and hence has prepared the following summary:

1. REVISIONS TO SEAOC "RECOMMENDED LATERAL FORCE REQUIREMENTS" (made since Appendix F was issued)

   a. Amend Section 2313 (b), DEFINITIONS, as follows:

      BOX SYSTEM - add "*or braced frames*" after shear walls.

      SHEAR WALL - delete second sentence concerning braced frames.

      Add new definition -
      BRACED FRAME is a vertical truss or its equivalent which is provided to resist lateral forces in the frame system and in which the members are subjected primarily to axial stresses.

   b. Amend Section 2313 (d) as follows:

      Section 2313 (d)1 modify first sentence as follows: *1. Total Lateral Force and Distribution of Lateral Force. Except as provided in Section 2313 (d)2,* every structure shall be designed and constructed to withstand minimum total lateral seismic forces _ _ _ _ _ _ _ .

      Add a new Paragraph 2313 (d)2:

      2. STRUCTURES HAVING IRREGULAR SHAPE OR FRAMING SYSTEMS. The distribution of the lateral forces in structures

b. (cont'd)

which have highly irregular shapes, large differences in lateral resistance or stiffness between different stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure.

Renumber Paragraphs 2 and 3 to 3 and 4 respectively.

c. Amend Section 2313 (i) as follows:

Modify the first paragraph as follows:

(i) Setbacks. Buildings having setbacks wherein the plan dimension of the tower in each direction is at least 75 percent of the corresponding plan dimension of the lower part may be considered as uniform buildings without setbacks *providing other irregularities, as defined in Section 2313 (d)2 do not exist. Other setback buildings shall conform to the provisions of Section 2313(d)2.*

d. Amend Section 2313 (j) as follows:

Section (j)1 - revise Paragraphs c and d

Paragraph 1c:

c. All concrete space frames required by design to be part of the lateral *force resisting system and all concrete frames located in the perimeter line of vertical support shall be space frames - ductile moment resisting.*

Exception: Frames in the perimeter line of vertical support of buildings designed with shear walls taking 100% of the design lateral forces need only conform with the following sub-item (d).

Paragraph 1d:

d. Change 2nd and 3rd lines to read: "resisting system shall be investigated ~~for~~ *and shown to be* adequate for vertical load carrying capacity *and induced moments due to* at four (4) times the distortions resulting from the code..."

e. Section (j)1 - add new Paragraph f.

f. All members in braced frames shall be designed for 1.5 times the force determined in accordance with Section 2313 (d)1. Connections shall be designed to develop the full capacity of the members or shall be based on stresses without the one-third increase usually permitted on those resulting from earthquake forces.

f. Amend Section 2313 (j)2, second and third paragraphs as follows:

~~Shear-walls-in-buildings-where-K-=-0.80~~ *Members of braced frames* shall be composed of ASTM A36, A440, A441, A572 (except Grades 60 and

f. (cont'd)

65) or A588 ~~Grades A, B, or C~~ structural steel; or reinforced concrete bracing members ~~or walls~~ conforming with the requirements of Section 2631 (b) of this Code.

Reinforced concrete shear walls ~~and reinforced concrete braced frames~~ for all buildings shall conform to the requirements of Section 2631 of this Code.

g. Amend Section 2313 (k) as follows:

Paragraph 2. fourth line, delete words ~~of this section~~

Paragraph 3. third line, put period after concrete. and delete words ~~under the provisions of Chapters 24 and 26~~.

Paragraph 3. sixth line, delete ~~using~~ and insert *having*.

h. Amend Table 23-C as follows:

For buildings with K=0.80, add in 3 places "or braced frames" after "shear walls." Delete footnote (4).

i. Amend Section 2630 (d)5a, as follows:

a. *Stirrups should be spaced to resist the ultimate design shear* V_u,

$$V_u \geqslant \frac{M_u^A + M_u^B}{L} + 1.4V_{D+L}$$

WHERE M_u^A and M_u^B ....

j. Amend Section 2630 (e)5 COLUMN SHEAR as follows:

$$A_v f_y \frac{d_c}{s} = V_u - V_c \qquad (30\text{-}8)$$

change $V_c = v_c bd$ to $V_c = v_c A_c$,

delete definition of d and add new definition

d_c = *dimension of column core in direction of load.*

k. Amend Section 2631 (b) as follows:

Delete words "in buildings with a ductile moment resisting space frame".

l. Amend Sections 2631 (d), and 2727 (c) by deleting words ~~Grades A, B, or C~~ after A588.

2. IMPORTANT REVISIONS TO 1970 EDITION OF UBC CONCERNING SEISMIC DESIGN.

There were many changes adopted by ICBO at its 1972 Annual Meeting to be included in the 1973 UBC. Some of the important changes made are presented below. Not all of these are in accord with the Seismology Committee's recommendations. Specifically, Appendix F to the Blue Book was adopted except the requirements for shear walls contained in ACI 381-71 were inserted in lieu of those specified in Appendix F.

a. Modify Section 2313 as follows:
Change existing second sentence to read:

Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this Chapter or a minimum force of 200 pounds per lineal foot of wall, whichever is greater.

Add the following paragraphs:

For floor and roof systems subject to lateral forces due to earthquake, cross ties, continuous between diaphragm chords, shall be provided. Such ties shall be capable, together with the associated wall anchorage, of resisting the forces tributary thereto. The distortions of the walls and the floor and roof system shall be considered in the design of the anchorage. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet.

Wood framing connected by toe nails or nails subject to withdrawal is not acceptable anchorage. Wood ledgers shall not be used in cross grain bending.

b. Amend Section 2314 (j) 1 to read as follows (many of the paragraphs are different from Appendix F): (j) **Structural Systems. 1. Design requirements.** All buildings designed with a horizontal force factor "*K*" = 0.67 or 0.80 shall have space frames-ductile moment resisting.

Buildings more than 160 feet in height shall have space frames-ductile moment resisting capable of resisting not less than 25 per cent of the required seismic forces for the structure as a whole.

EXCEPTION: Buildings more than 160 feet in height in Seismic Zone No. 1 may have concrete shear walls designed in conformance with Section 2627 of this Code in lieu of a ductile moment resisting space frame, provided a "*K*" value of 1.00 or 1.33 is utilized in the design.

In Seismic Zones #2 and #3, all members in braced frames of K=1.0 and K=1.33 buildings, shall be designed for 1.5 x the face determined in accordance with Section 2314 (d) 1.

Connections for these members are not permitted the 33% stress increase for earthquakes.

In Seismic Zones #2 and #3, concrete space frames required by design to be part of the lateral force resisting system and all concrete frames located in the perimeter line of vertical support shall be space frames-ductile moment resisting.

EXCEPTION: Frames in the perimeter line of the vertical support of buildings designed with shear walls taking 100 per cent of the design lateral forces need only conform with the following paragraph:

In Seismic Zones #2 and #3, framing elements not required by design to be part of the lateral force resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity and induced moment due to four times the distortions resulting from the Code required lateral forces. The rigidity of other elements shall be considered in accordance with Section 2314 (c).

Moment resisting space frames and ductile moment resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

Other structural concepts may be approved by the Building Official when evidence is submitted showing that equivalent ductility and energy absorption are provided.

Note: 2314 (j)2 also revised per SEAOC Blue Book.

c. Amend Table 23-J by adding the following:

TABLE NO. 23J — HORIZONTAL FORCE FACTOR "C_p" FOR PARTS OR PORTIONS OF BUILDINGS OR OTHER STRUCTURES

PART OR PORTION OF BUILDINGS	DIRECTION OF FORCE	VALUE OF C_p
When connected to, part of, or housed within a building: towers, tanks, towers and tanks plus contents, storage racks over 6 feet plus contents, chimneys, smokestacks and penthouses	Any direction	.20[2, 4]
Suspended ceiling framing systems[5]	Any horizontal direction	0.20

2 When located in the upper portion of any building where the "h_n/D" ratio is five-to-one or greater the value shall be increased by 50 per cent.

4 "W_p" for storage racks shall be the weight of the racks plus contents. The value of "C_p" for racks over two storage support levels in height shall be .16 for the levels below the top two levels.

5 For purposes of determining the lateral force, a minimum ceiling weight of 5 pounds per square foot shall be used. Applies to Zones 2 and 3 only.

d. Revise definition of "W" in Section 2314 (c) to read as follows:

W = Total dead load as defined in Section 2301 including the partition loading specified in Section 2303 (b) where applicable. (Exception remains).

e. Section 2314 (d), modify third paragraph as follows:

For all one- and two-story buildings or structures the value of C shall be not less than 0.10. For other buildings the maximum value of C need not exceed 0.10.

Exceptions:
1. C exceeds 0.10 where indicated in Table No. 23-1.
2. Buildings or structures which have highly irregular shapes, large differences in lateral resistance or stiffness between different stories or other unusual structural features affecting seismic response shall be designed for forces which their dynamic properties induce.

f. Table No. 24-H, change shear provisions and add footnotes 5 and 6 to read as follows (Note: The revision to Table 24-H was proposed by the Masonry industry and as an interim measure the Seismology Committee recommended adoption of footnote 5 to the Table).

f. (cont'd)

TABLE NO. 24-H — MAXIMUM WORKING STRESSES IN POUNDS PER SQUARE INCH FOR REINFORCED SOLID AND HOLLOW UNIT MASONRY[1]

TYPE OF STRESS	SPECIAL INSPECTION REQUIRED Yes	No
Shear:		
No shear reinforcement,		
Flexural[5]	$1.1\sqrt{f'_m}$ 50 Max.	25
Shear walls		
$M/Vd \geq 1$[6]	$.9\sqrt{f'_m}$ 34 Max.	17
$M/Vd = 0$	$2.0\sqrt{f'_m}$ 50 Max.	25
Reinforcing taking all shear,		
Flexural	$3.0\sqrt{f'_m}$ 150 Max.	75
Shear walls[5]		
$M/Vd \geq 1$[6]	$1.5\sqrt{f'_m}$ 75 Max.	35
$M/Vd = 0$	$2.0\sqrt{f'_m}$ 120 Max.	60

5. When calculating shear stress, shear walls which resist seismic forces shall be designed to resist two times the forces required by Section 2314 (d) 1.

6. Interpolate by straight line for M/Vd values between 0 and $\geq$ 1.

g. Section 2314 (c). All of the State of California is now included in Zone 3 (will be shown on new seismic zoning map).

h. Section 1807 (k). Add new paragraph to cover Type 1 and 2 buildings.

(k) Seismic Considerations. In Seismic Zones 2 and 3 the anchorage of the following mechanical and electrical equipment required by the Section shall be designed in accordance with Section 2314 for a lateral force based on a "Cp" value of 0.5 unless data substantiating a lesser value is furnished:

1. Elevator drive and suspension systems.
2. Standby power and lighting facilities.
3. Fire pumps and other fire protection equipment.

3. ITEMS CURRENTLY UNDER STUDY BY SEISMOLOGY COMMITTEE

The SEAOC Seismology Committee in addition to other items is currently studying the following with the goal of developing appropriate Code provisions.

a. Site seismicity or seismic exposure.
b. Effects of soil-structure interaction.
c. Dynamic design principles.
d. Importance factors for structures.
e. Evaluation and upgrading of potentially hazardous existing buildings and structures.

APPENDIX V - DISSEMINATION OF RESEARCH RESULTS. This symposium is an example of how research results on dynamic analysis and design of structures are transmitted to the engineering profession.

"EARTHQUAKE SYMPOSIUM"

WEDNESDAY, JUNE 13, 1973 – 4 to 6 P.M.

UNION OIL AUDITORIUM
461 S. BOYLSTON
LOS ANGELES, CALIF.

Sponsored by
The Steel Committee
American Institute of Steel Construction

PROGRAM PHILOSOPHY

This Earthquake Symposium is planned to inform you of the latest developments in seismology within the Structural Engineers Association of California, and the Structural Steel Industry. In addition to presentations on recent research performed at leading universities, progress reports will be given by the various Seismology Committees of the Structural Engineers Association of Southern California. Attesting to the Committee's activity is Appendix F-1969 through 1971 revisions to the SEAOC Lateral Force Requirements, which has recently been adopted by the Los Angeles City and Uniform Building Codes. You will also hear the consulting engineer's opinion regarding the current Code and its requirements for a dynamic response analysis. Of course, cost is always a factor to be dealt with and the recent Code changes do affect the cost. How much, you might ask? Plan to attend our Symposium and learn the answer and much more.

TOPICS AND SPEAKERS

Moderator, Jim Marsh, AISC Regional Engineer

COMMENTARY ON THE AMENDED SEAOC RECOMMENDED LATERAL FORCE REQUIREMENTS.
Don Strand, Chairman of SEASC Seismology Committee

EARTHQUAKE RESISTANT DESIGN OF BUILDINGS EMPLOYING DYNAMIC RESPONSE ANALYSIS . . . Introduction of basic concepts for dynamic response analysis relating its current use to present lateral force code requirements. Application of a dynamic response analysis to the Metropolitan Life Building of San Francisco.
Stephen Johnston, Structural Engineer, Skidmore, Owings & Merrill

SEISMIC INSTRUMENTATION AND ITS ROLE IN EVALUATING THE RESPONSE OF STRUCTURES
John Robb, Chairman of SEASC Seismology Sub-Committee on Seismic Instrumentation and Testing

DESIGN OF BRACED FRAME BUILDINGS . . . Introductory background to the analytical behavior of bracing members based on results of current research. Design recommendations of X-braced frames and results of inelastic response studies of K-braced frames.
Robert D. Hanson, Associate Professor of Civil Engineering, University of Michigan

EFFECT OF SOILS ON SEISMIC ANALYSIS OF BUILDINGS
David Leeds, Chairman of SEASC Seismology Sub-Committee on Foundation Interaction Response

COST EFFECTS OF RECENT CODE PROVISIONS . . . A review of cost effects (based on somewhat limited experience) that the recent Code changes, particularly with regard to the dynamic analysis requirement, are having on structural steel frame costs.
Stewart Easterby, Chairman of AISC STEEL Committee

SPEAKERS

STEWARD D. EASTERBY
Sales Engineer
Bethelehem Steel Corporation
Los Angeles

ROBERT D. HANSON
Associate Professor of Civil Engineering
University of Michigan
An Arbor, Michigan

DAVID J. LEEDS
Engineering Seismologist
Dames and Moore
Consultants in Applied Earth Sciences
Los Angeles

STEPHEN E. JOHNSTON
Associate Partner
Skidmore Owings and Merrill
Architects/Engineers, San Francisco

JOHN O. ROBB
Senior Structural Engineer
Los Angeles City, Department of Building and Safety, Research Bureau

DONALD R. STRAND
Associate, Brandow & Johnston, Associates

PARKING:

FEE: None (a bargain at today's prices)

PARKING: The Los Angeles Chamber of Commerce parking lot (404 S. Bixel) which is one block from the Union Oil Auditorium will be available for parking. Enter from Fourth Street. Parking fee, 75¢.

APPENDIX VI - EARTHQUAKE RISK ANALYSIS

Example, showing how research is put into practical application. Research done at the California Institute of Technology, on NSF-sponsored programs, was used for assessing the earthquake risk for high-rise buildings in Los Angeles, and then was used by practicing engineers.

"What is Sea Grant?" .... **Page 3**
Speaker: RONALD B. LINSKY
Director, Sea Grant Program, University of Southern California
Marine Environment and Resources Section

"Shake, Rattle, and Roll—When the Earth 'Rocks'" .... **Page 4**
Speaker: LeROY CRANDALL
President, LeRoy Crandall and Associates
Systems and Technology Section

"Denmark and the Common Market" .... **Page 6**
Speaker: PETER KNOP
President, Danish Engineering Society, Denmark
International Relations and Economics Sections

"The Role of the Action Agency in the Business Community" .... **Page 8**
Speaker: JOHN A. BUTLER
Regional Director, Action, Region IX
Town Hall West Special Section

"Chicano Power" .... **Page 10**
Speaker: DAVID SANCHEZ
Former Prime Minister, The Brown Berets
Race Relations Section

"The Southern California Regional Aviation Systems Study" .... **Page 11**
Speaker: WALTER E. GILLFILLAN
Study Director, Regional Airport Systems Study, Southern California Association of Governments (SCAG)
Transportation and Regional Planning & Development Sections

"The Quality of Life" .... **Page 13**
Speaker: HAROLD KLEIN
President, Urbanetics Financial Corporation
Town Hall West Economics Section

"ERA—Taking the Financial Jolt Out of an Earthquake" .... **Page 14**
Speaker: BOB NEW
President, Bob New, Inc.
Systems and Technology Section

"A Traveler's Notes on White Africa" .... **Page 15**
Speaker: REX J. L. HEYMANN
Registered Representative and Security Analyst, Sutro & Co.
International Relations Section

"Los Angeles County General Plan Program" .... **Page 18**
Speaker: JOSEPH K. KENNEDY
Deputy Director, Long Range Planning, Regional Planning Commission, Los Angeles County
Regional Planning and Development Section

Cover Picture: Six ships being worked in the deep-water "Indies Terminal" at the Port of Los Angeles—framed by the spans of the Vincent Thomas Bridge.

TOWN HALL is an impartial forum dedicated to civic education and to the discussion of public questions.

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"ERA—Taking the Financial Jolt Out of an Earthquake"

Speaker: BOB NEW
President, Bob New, Inc.

Page 14/Town Hall Reporter

Our high-rise skyline and the memory of earthquakes raise questions in many minds. How safe are the high-rise buildings? In fact, how safe is any building in a major earthquake? Bob New, as an insurance consultant, has spent many years seeking solutions to the problems of earthquake risk. Impressed with Albert C. Martin and Associates success in computerized earthquake simulation techniques, Mr. New has translated the technical data into easy-to-follow guidelines and now has exclusive rights to the service—called Earthquake Risk Analysis.

Presented to Systems and Technology Section
William A. Thacker, Chairman

". . . and, lo, there was a great earthquake . . . and every mountain and island were moved out of their places." So relates the Biblical book of the Revelation of St. John. Earthquakes were news then, but not new. Earthquakes remain news. Since much of the world must live with them, since we cannot accurately predict them, how then can we best prepare for them? One answer is systematic analysis—Earthquake Risk Analysis (ERA). Originally developed and tested by Albert C. Martin and Associates, it uses space-age techniques to replace trial and error—sometimes fatal error.

Each of us lives and works in buildings. And each of us wonders how the structure will react in a serious earthquake. To find the answers, a computer was programmed to analyze what would happen to every structural member and joint in a building subjected to simulated earthquake stresses. The stresses used ran from 4.5 to 8.5 magnitude on the Richter Scale. Base data, insofar as possible, were extracted from actual recorded earthquake damage—and survival.

As we all know, Southern California lies over a series of earthquake faults. An irregularly arranged set of "time bombs." However, most of us are not aware that there are significant faults in other areas of the United States. Few people realize that one of the most violent earthquakes ever experienced in North America occurred in New Madrid, Missouri, in 1812. Other states, too, have their earthquake history; thus, ERA has local, national, and worldwide importance.

ERA

How does ERA actually work? First comes the gathering of great masses of data—principally dealing with fault lines, soil conditions, and all available engineering data. With this data the building is "built" in the computer on the chosen site. The "building" then is submitted to a wide range of "earthquake jolts." The resulting stresses and distortions are evaluated during the entire period of the earthquake. The results are printed out and recast in layman's language.

The next step becomes judgmental. The computer can supply the risk analysis, but the owner of an existing or planned building must then decide what steps to take.

In a building under construction, measures can be taken to strengthen any weak spots—but to what level? With older buildings, the options are reduced but the decisions must still be made.

Perhaps the clearest way to describe one kind of decision that must be made is by case study. In one instance, an analysis of a $5 million manufacturing plant was made. The computer predicted a 90% chance of $10,000 in damages, a 40% chance of $50,000 in damages, and a 4% chance of $800,000 in damages. The owner had been paying an earthquake insurance premium of $54,000 annually, and the policy had a $1 million deductible clause. After considering the probabilities and circumstances, the owner decided to underwrite his own insurance. Why pay $54,000 for a $1 million deductible policy when the chances of such a loss are 96 to 4? ERA evaluations are equally helpful if you plan to buy a building, to loan money on one, or even rent space.

ACCURACY

ERA was developed prior to the February 9, 1971, San Fernando Valley earthquake. Several major buildings had been subjected to analysis prior to the earthquake and the uncanny accuracy of the dynamic simulation techniques was most impressive. Among the buildings analyzed were the Los Angeles Department of Water and Power building, the Arco Plaza Twin Towers, Sears headquarters building in Alhambra, the Union Bank building in downtown Los Angeles, and the Sunkist building in Sherman Oaks. After the February earthquake, a comparison of predicted and actual responses was made and when portrayed in graph form the two lines almost exactly coincide.

Application of the ERA service to an existing building can supply information that engineers can use to devise reinforcements where needed and thus create a safer structure less likely to be damaged in a severe earthquake. Buildings are constructed with "predictable" materials, under specific code provisions, and usually the blueprints and specifications are available. With this amount of data the analysis can be quite specific in finding solutions to reinforce any potential weak points. There also is a strong monetary, as well as humanitarian, motive for ERA since the strengthening reduces damage and consequent business interruption or losses of rental income. Also substantial insurance savings are possible. Above all, in the case of planned buildings, the owners and occupants can see how the completed structure would stand during an earthquake *before* construction and revise their plans accordingly.

QUESTIONS & ANSWERS

Q. We are tenants of the Arco Towers. You said an analysis had been made. How do we find out what would happen to our offices and what safeguards we should take?
A. ERA provides information on how various pieces of equipment would move as the building responds to an earthquake. The owners should provide your firm with that information.

Q. In developing ERA how much of the information used in the analysis was based upon actual experience and how much upon engineering theory?
A. About 6 years ago Albert C. Martin and Associates became interested in the computer simulation possibilities. They sent a representative to various countries that had long histories of earthquake damage. Much to their surprise, they learned that Caltech and other organizations in Southern California were far more sophisticated about earthquakes than experts anywhere else. Using available information, they developed their theories and techniques. They also worked out the programming method of subjecting the "structure" in the computer to "earthquakes." Although as much as possible of their base information came from actual earthquake damage, it was the earthquake of February 9 that was the real test. It also gave them a tremendous amount of new data.

Q. Is there any attempt being made to include ERA in the code requirements?
A. Yes. At present it is requested but not required. We believe that it will be required soon.

Q. Is ERA applicable to concrete construction as high as 6 stories?
A. Yes. We are analyzing a hospital now.

Q. Has ERA found any major safety differences between steel construction and reinforced concrete?
A. From the standpoint of earthquakes, steel buildings are safer. Most dangerous is concrete block.

JoAnn Wohlstattar, Secretary
Systems and Technology Section